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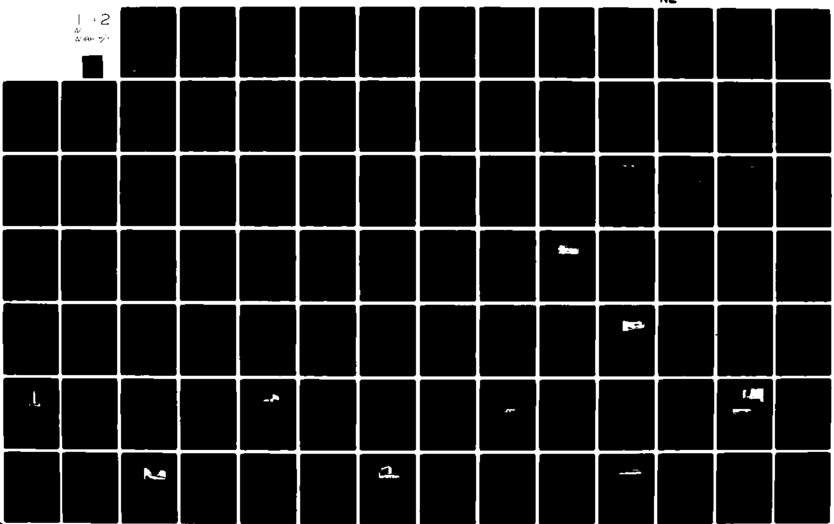

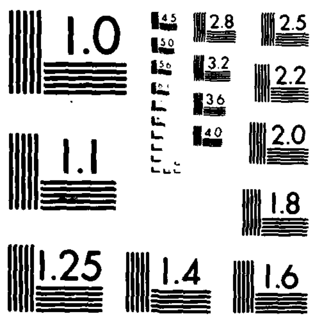

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Final Report

July 1980

Prepared for:

FEDERAL EMERGENCY MANAGEMENT AGENCY
WASHINGTON, D.C. 20472

Contract DCPA01-78-C-0295
FEMA Work Unit 1151D

(SRI International Project HSU-7850)

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Dynamic Analysis	Building Analysis	Waffle Slab
Floor-Over-Basement	Wall	Flat Plate
Blast Shelter	Beam	Flat Slab
Upgrading	Slab	Single-Degree-of-Freedom Model

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report covers the collapse analyses of floor-over-basement areas. The floors were separated into floor systems and were analyzed "as built" and for various upgrading configurations through an examination of individual elements. The purpose of the report is two-fold: first, to increase the data base of analyzed "as built" NSS building floors; and second, to determine the expedient upgrading potentials of NSS building floors.

This report summarizes the results of the collapse analyses of the 11 NSS buildings examined in this study. The results of the "as built" analyses

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19. KEY WORDS (Continued)

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are then grouped with the collapse analyses of 36 NSS buildings to provide a population of 46 buildings (one building was reexamined). The predicted collapse overpressures, examined previously by Wiehle (1974), of the weakest floor element by building and by floor system are presented in the form of histograms and cumulative frequency distributions. The effect of frame type on the collapse strength of the floor elements was examined as in the previous report (Wiehle, 1974).

This report also summarizes, for the 11 buildings analyzed herein, the upgrading potentials of floor elements grouped by individual element, floor system and building. Preliminary indications of these collapse analyses indicate that the best way to assess which building and/or element is most upgradable is to look for elements, especially slabs or pan-joist systems, having the greatest span (i.e., span lengths great enough to allow intermediate supports at third- or even quarter-span intervals). Analytical results indicate that if upgrading is accomplished by putting intermediate supports at 1.8 m (6 ft) spacing, a reinforced concrete slab with a thickness of 150 mm (6 in.) can resist an overpressure of 210 kPa (30 psi) and a slab with a thickness of 200 mm (8 in.) can resist 350 kPa (50 psi). Long spans usually assure slab thicknesses of 150 mm (6 in.) or more.

The appendices include information on a comparison of the SRI analytical procedures with the results of recent U.S. Army Engineer Waterways Experiment Station (WES) tests on "as built" and upgraded floor systems, and an analysis of the effect of soil mass on the response of buried basement walls to air blast.

Wiehle, C. K., "Summary of the Dynamic Analysis of the Exterior Walls and Floor Systems of 50 NFSS Buildings," SRI (for DCPA), Menlo Park, California, June 1974.

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This report summarizes the results of the collapse analyses of the 11 NSS buildings examined in this study. The results of the "as built" analyses are then grouped with the collapse analyses of 36 NSS buildings to provide a population of 46 buildings (one building was reexamined). The predicted collapse overpressures, examined previously by Wiehle (1974), of the weakest floor element by building and by floor system are presented in the form of histograms and cumulative frequency distributions. The effect of frame type on the collapse strength of the floor elements was examined as in the previous report (Wiehle, 1974).

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[1]SI units are used for this report. English units are given for reference only and are "rounded off" for convenience. This rounding is denoted by "≈". Where rounding to eliminate decimals was not used, the converted numbers were given to the same number of significant figures as the SI values.

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Chapter 1

INTRODUCTION

Under contract with the Federal Emergency Management Agency and in support of the all-effects shelter survey system, SRI performed dynamic analyses of the floor-over-basement areas of actual NSS buildings. These floor-over-basement areas, or simply floors, were separated into one or more floor systems. It was the individual elements comprising the floor systems on which the collapse analysis was directly performed. Sensitivity analyses were subsequently performed to investigate what effect reducing the clear spans of these elements would have on their predicted collapse overpressure. This report summarizes the results of these analyses.

1.1 SCOPE

As part of an integrated program to develop a procedure for an all-effects shelter survey system, Research Triangle Institute (RTI), collected data on a national sample of 219 NSS facilities (Tolman, Lyday & Hill, 1973). In previous efforts, SRI analyzed the exterior walls and the floor-over-basement areas of 36 buildings to determine their collapse overpressure when subjected to nuclear air blast loading (summarized in Wiehle, 1974).

During the current effort, SRI has analyzed the floor-over-basement areas of the following 11 NSS buildings:

NSS Building Number	Description
100	U.S. Post Office, Harrisburg, PA
110	Henry R. Landis State Hospital, Philadelphia, PA
111	Grant Building, Pittsburgh, PA
136	First Federal Savings & Loan Assn., Augusta, GA
167	Lafayette Towers Building #2, Detroit, MI
188	State Wildlife Conservation Building, Oklahoma City, OK
200	Fitzsimmons General Hospital, Denver, CO
220	Fidelity Federal Plaza, Long Beach, CA
225	Broadway Crenshaw Building, Los Angeles, CA
227	May Company Shopping Center, West Covina, CA
245	Portland Hilton Hotel, Portland, OR

The analysis of each structure was based on information obtained from the field survey data collection forms, sketches, photographs, and building plans, all furnished by RTI. If a discrepancy was found in the information

from these sources, the building plans were the source used in the analysis. Each of the floor elements was also analyzed assuming one or more upgrading schemes.

1.2 DISCUSSION AND REPORT ORGANIZATION

For this effort, the floor-over-basement areas of each building were analyzed using the available mathematical models and computer programs developed in the SRI/DCPA research program reported in Wiehle and Bockholt (1968, 1970, & 1973). The dynamic loading used was equivalent to the free-field overpressure created by a 1-Mt surface burst, except with a rise-time equal to the travel time of the wave front across the floor panel.

The results of the analyses of the 11 buildings in the current effort are tabulated in this report; however, for the statistical comparison, the results of the floor analyses of 35 buildings (80 floor systems) summarized in Wiehle (1974), were added to the present sample. The discussion of these "as built" analyses is given in Chapter 2.

The second part of this report deals with the upgrading potential of the 11 NSS buildings. Various schemes were used to try to determine the maximum upgrading potential of floor elements and systems. The results of these schemes are given in Chapter 3. Only the upgrading potential of the floors was considered. Neither the transfer of load from the upgraded system to the foundation wall nor the strength of the exterior basement wall were investigated in this research effort.

Appendix A gives an example of the procedure used to analyze a structure. It outlines the steps involved in gathering information necessary for making a response calculation, using the first building for illustration. Appendix B consists of brief descriptions of the remaining buildings examined and discussions of the "as built" and upgraded results for each. Appendix C describes a comparison made between SRI SDOF[2] models and recent dynamic laboratory (unpublished) tests conducted by the U.S. Army Engineer Waterways Experiment Station (WES) on "as built" and upgraded floor systems. Appendix D contains results of a buried exterior basement wall analysis. The analysis was performed to show what effect different soil masses, which responded with the wall, would have on the predicted mean incipient collapse overpressure (MICO) of the wall. Appendix E gives the main summary table of the report in English units. Appendix F contains the notation.

1.3 ACKNOWLEDGEMENTS

The author gratefully acknowledges the assistance and guidance of G. N. Sisson and M. A. Pachuta of FEMA. Also acknowledged are B. M. Beaver of SRI and H. L. Murphy of H. L. Murphy Associates for their contributions to this effort.

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[2]Single-degree-of-freedom model.

Chapter 2

BUILDING ANALYSIS OF THE "AS BUILT" CASE

Each building examined followed a set procedure. A step-by-step example showing how the analysis was carried out is given in Appendix A using Building 100 as an illustrative case. The remaining cases are presented in Appendix B. For each of these a description, photograph, outline plan of the first floor and location of floor systems studied, "as built" analysis, and upgrading potential analysis are given. This chapter discusses the results of the analysis performed on the "as built" cases.

2.1 ANALYSIS

This section is broken into three parts: The first summarizes results with respect to the individual floor elements. The second summarizes results with respect to floor systems; in this report floor systems refer to structural assemblages consisting of one or more floor elements (slabs, beams, girders, flat plates, or flat slabs), which are representative of part or all of a whole floor-over-basement area. The third part summarizes results with repect to the overall buildings.

2.1.1 Floor Element Analyses

For each building studied, the basic units of analysis were the elements constituting each floor system. Analyses of the 11 buildings in the present study required the calculation of incipient collapse overpressures of a flat slab, a flat plate, 17 floor slabs, and 25 support beams (including joists and girders), within 19 floor systems. The results of the floor element analyses are summarized in Table 1. (Table 1 is in metric units; an English version is in Appendix E.)

Each of the 11 buildings are listed in Table 1 according to NSS building number. The type of frame used in the building is given in parentheses following the building title. RCF refers to reinforced concrete (R/C) frame, STF to steel frame. In this study there are ten R/C frame structures and one steel frame structure.

The columns "Case" and "Element Type" in Table 1 identify the element being studied. Under "Case," each number designates a separate el-

ement; the letter following gives the upgrading scheme. The "a" or "A" case refers to the "as built" case. The other letters refer to upgrading schemes which will be discussed in Chapter 3, except for "/r" and "/s", which are discussed further below. Lower case letters refer to slabs while capital letters indicate beams, joists or girders. Using Building 100 for illustration, seven elements are shown: five slabs, one beam and one joist.

The key to the entries under "Element Type" is given in Table 2. The number following the element type in Table 1 identifies the floor system to which the element belongs, e.g., RCB-1 refers to a beam in floor system #1. In Building 100 there are 5 floor systems: system #1 contains one reinforced concrete (R/C) slab, system #2 contains one R/C slab and one R/C beam, system #3 contains one R/C slab, system #4 contains one R/C slab and one R/C joist, and system #5 contains one R/C slab.

The columns "L_s", "L_L", "b_b", "h", "f'_c", "f_{dy}", "Reinforcing Ratios", and "Tensile Membrane Steel" give structural properties of the element. Descriptions of these engineering terms are given in Appendix F. The column "Support Case" gives the support conditions; Table 2 gives the key. The final four columns under the heading "Incipient Collapse Overpressure" provide the results of each analysis. The mean value of the incipient collapse overpressure (MICO) may be taken as the overpressure at or below which failure will occur 50 percent of the time. "10% Prob." and "90% Prob." refer to values for which failure is predicted to occur 10 and 90 percent of the time respectively.

For flat plate or flat slab systems, a "/s" in the case number indicates shear failure was included in the calculation. Experience has shown that shear failure normally does not control the final dynamic failure mechanism when there is adequate anchorage to assure full development of the tensile membrane mode. The "/s" case should be used for comparison when there is no tensile membrane steel. Both cases, however, are given for completeness since there can be a substantial difference in the calculated incipient collapse overpressure depending on the assumption made.

In pan-joist systems, the slab cases were calculated using two models, the regular slab model and a restrained model. Those cases designated by a "/r" were analyzed as being fully restrained from horizontal motion. The prediction resulting from the restrained model is felt to be correct in most situations since the floor system should supply full restraint to the slab. The regular case (without "/r") was included for completeness and to predict those situations where full restraint does not exist, e.g., an exterior panel without a beam for closure or a prefabricated section.

2.1.1.1 Slabs/Flat Plates

The 19 floor systems contained a flat slab, a flat plate, three slabs in slab floor systems (wall supports only), nine slabs in slab-and-beam floor systems, and five slabs in pan-joist floor systems. The mean incipient collapse overpressures (MICOs) for all slabs ranged between 22.3 kPa (3.2 psi) and 1297.1 kPa (188.1 psi). Slabs in the pan-joist systems, which were analyzed as fully restrained, had MICOs between 355.1 kPa (51.5 psi) and 1297.1 kPa (188.1 psi). The highest slab MICO outside of a pan-joist system was 94.1 kPa (13.6 psi).

2.1.1.2 Beams/Joists/Girders

The 25 beams, joists, and girders analyzed were found in 14 of the 19 floor systems. The MICOs ranged between 24.1 kPa (3.5 psi) and 126.5 kPa (18.4 psi). The highest MICO in the beam range (126.5 kPa) is far below the highest MICO in the slab range (1297.1 kPa). If the slabs in pan-joist systems are excluded, the slab MICO range (22.3 - 94.1 kPa) appears lower than the beam MICO range (24.1 - 126.5 kPa); however, the slab is the weakest element in only three of the 14 floor systems having beams, joists, or girders.

2.1.1.3 Tensile Membrane Behavior

An analysis of the importance of tensile membrane was performed for those cases where tensile membrane behavior was expected, i.e., those cases having values in the "Tensile Membrane" column of Table 1. An asterisk (*) in the "Mean Incipient Collapse Overpressure" column indicates that the tensile membrane mode controlled in the calculation of the incipient collapse overpressure. No asterisk means the tensile membrane behavior did not control the calculated incipient collapse overpressure. As can be seen from the table, tensile membrane behavior seldom sets the maximum resistance in most cases. There also does not appear to be a correlation between those cases controlled by the tensile membrane mode and the percent of tensile membrane steel in the element. It should be noted, however, that the one building (Building 225) containing a flat slab system with tensile membrane steel had tensile membrane behavior controlling the incipient collapse overpressure in all the scenarios studied. This would indicate that continuity and proper anchorage of steel in flat plate or flat slab systems is particularly important in assuring full development of the resistive capacity of the plate or slab.

2.1.2 Floor System Analyses

The 19 floor systems in the current study were estimated to be the weakest in their particular floors. The individual elements of the systems were analyzed using SRI's SDOF models, and the individual element with the lowest mean incipient collapse overpressure (MICO) determined the collapse potential of the floor system of which it was a part, the floor system only being as strong as its weakest element. The predictions for each floor system are given in Table 3 by building. Also shown is the percentage each system represents of the total floor-over-basement area of the building.

For statistical comparison of the floor systems, the results of the analyses of the 19 floor systems were added to the results of 80 other floor systems summarized in Wiehle (1974), providing a total population of 99 floor systems. The results of the dynamic analyses of all floor cases are summarized in the histogram and cumulative frequency distribution shown in Figure 1. As indicated on the figure, the collapse overpressure for the floor systems ranged from 17.2 to 373.0 kPa (2.5 to 54.1 psi), with 50 percent of the systems predicted to collapse at 47 kPa (6.8 psi) or less, and 90 percent predicted to collapse at 119 kPa (17.3 psi) or less.

2.1.3 Building Analyses

Figure 2 shows a histogram and frequency distribution of the MICO of the 46 buildings represented by the 99 floor systems above. For purposes of analysis the collapse potential for each building is based on the lowest collapse potential of the floor systems comprising its floor-over-basement area. In reality, however, one floor system might collapse without collapsing the entire floor area. The median MICO by building (Figure 2) is 35 kPa (5.1 psi), which is 26 percent lower than would have been predicted by looking at floor systems only (Figure 1).

TABLE 1

SUMMARY OF FLOOR ELEMENT ANALYSES

Case	Element Type	L_s (m)	L_L (m)	b_b (mm)	h (mm)	f_c' (MPa)	f_{dy} (MPa)	Support Case	Reinforcing Ratios at Cross Sections: 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio: Short	Long	Incipient Collapse Overpressure: Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)
Building 100, U.S. Post Office (RCF)																						
1a	RCS-1	8.560	33.528		304.8	20.68	365.42	1	0.0172	0.0013	0.0013	0.0013	--	--	--	--	0.0013	--	39.23	4.34	33.65	44.75
1b	RCS-1	4.280	33.528		304.8	20.68	365.42	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	158.99	19.03	134.59	183.40
1c	RCS-1	2.852	33.528		304.8	20.68	365.42	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	353.22	31.44	312.88	393.55
1d	RCS-1	2.140	33.528		304.8	20.68	365.42	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	630.39	59.50	554.13	706.64
2a	RCS-2	2.013	3.327		114.3	20.68	365.42	4	0.0051	0.0051	0.0039	0.0	0.0048	0.0048	--	--	0.0051	--	*94.05	11.17	79.77	108.39
2e	RCS-2	2.013	1.664		114.3	20.68	365.42	4	0.0051	0.0051	0.0039	0.0	0.0048	0.0048	--	--	0.0051	--	116.66	13.58	99.29	134.04
3A	RCB-2	5.004		1019.0	419.1	20.68	365.42	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	*126.52	16.96	104.74	148.31
3B	RCB-2	2.502		1019.0	419.1	20.68	365.42	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	*268.97	37.10	221.47	316.48
4a	RCS-3	7.341	50.292		304.8	20.68	365.42	5	0.0139	0.0	--	--	--	--	--	--	--	--	40.20	[illegible].28	34.68	45.64
4b	RCS-3	3.670	50.292		304.8	20.68	365.42	5	0.0139	0.0	--	--	--	--	--	--	--	--	176.51	18.41	152.93	200.15
4c	RCS-3	2.446	50.292		304.8	20.68	365.42	5	0.0139	0.0	--	--	--	--	--	--	--	--	410.03	47.02	349.70	470.36
4d	RCS-3	1.835	50.292		304.8	20.68	365.42	5	0.0139	0.0	--	--	--	--	--	--	--	--	743.88	83.22	637.28	850.54
5a	RCS-4	0.449	2.096		76.2	20.68	365.42	6	0.0072	0.0	--	--	0.0072	0.0	--	--	0.0072	--	286.49	30.96	246.84	326.13
5a/r	RCS-4	0.449	2.096		76.2	20.68	365.42	n/a	0.0072	0.0	0.0012	0.0	0.0072	0.0	0.0012	0.0	0.0072	--	1297.11	130.66	1164.04	1464.58
6A	RCJ-4	9.144		184.9	431.8	20.68	365.42	5T	0.0203	0.0	--	--	--	--	--	--	--	--	24.20	3.52	19.72	28.75
6B	RCJ-4	4.572		184.9	431.8	20.68	365.42	5T	0.0203	0.0	--	--	--	--	--	--	--	--	109.08	13.65	91.57	126.52
6C	RCJ-4	3.048		184.9	431.8	20.68	365.42	5T	0.0203	0.0	--	--	--	--	--	--	--	--	237.67	22.75	208.57	266.84
6D	RCJ-4	2.286		184.9	431.8	20.68	365.42	5T	0.0203	0.0	--	--	--	--	--	--	--	--	436.80	51.23	371.09	502.44
7a	RCS-5	5.664	59.131		254.0	20.68	365.42	5	0.0100	0.0	--	--	--	--	--	--	--	--	35.58	4.96	29.23	41.92
7b	RCS-5	2.832	59.131		254.0	20.68	365.42	5	0.0100	0.0	--	--	--	--	--	--	--	--	156.65	19.51	131.63	181.68
7c	RCS-5	1.887	59.131		254.0	20.68	365.42	5	0.0100	0.0	--	--	--	--	--	--	--	--	347.03	32.27	305.66	388.40
Building 110, Henry R. Landis State Hospital (RCF)																						
1a	RCS-1	4.343	39.421		177.8	22.75	365.42	7	0.0074	0.0	--	--	0.0087	0.0	--	--	--	--	34.89	3.72	30.13	39.65
1b	RCS-1	2.172	39.421		177.8	22.75	365.42	5	0.0074	0.0	--	--	--	--	--	--	--	--	93.50	9.24	81.64	105.29
2A	RCB-1	6.706		457.2	508.0	22.75	365.42	6	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	28.61	3.93	23.58	33.65
2B	RCB-1	3.353		457.2	508.0	22.75	365.42	7	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	86.74	8.41	75.98	97.50
2C	RCB-1	2.235		457.2	508.0	22.75	365.42	5	0.0127	0.0	--	--	--	--	--	--	--	--	120.39	14.07	102.39	138.38
3a	RCS-2	4.432	6.452		152.4	22.75	365.42	3	0.0037	0.0	0.0030	0.0	--	--	0.0042	0.0027	--	--	22.27	4.21	16.96	27.65
3b	RCS-2	2.216	6.452		152.4	22.75	365.42	1	0.0037	0.0	0.0030	0.0	--	--	--	--	--	--	46.68	6.48	38.34	54.95
4A	RCB-2	4.216		558.8	355.6	22.75	365.42	7	0.0093	0.0	--	--	0.0157	0.0094	--	--	--	--	41.16	4.62	35.30	47.09
4B	RCB-2	2.108		558.8	355.6	22.75	365.42	5	0.0093	0.0	--	--	--	--	--	--	--	--	97.15	9.45	84.95	109.29
5a	RCS-3	2.896	6.502		101.6	22.75	365.42	6	0.0111	0.0	--	--	0.0097	0.0056	--	--	0.0056	--	*53.71	5.72	46.40	61.09
6A	RCB-3	6.604		508.0	304.8	22.75	365.42	6	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	*31.92	4.55	26.06	37.85
6B	RCB-3	3.302		508.0	304.8	22.75	365.42	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	90.74	9.03	79.22	102.25
6C	RCB-3	2.201		508.0	304.8	22.75	365.42	5	0.0291	0.0	--	--	--	--	--	--	0.0099	--	117.63	10.76	103.84	131.42
6E	RCB-3	3.302		508.0	304.8	22.75	365.42	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	109.29	8.69	98.12	120.39
7A	RCG-3	4.953		711.2	609.6	22.75	365.42	6	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	28.96	3.31	24.75	33.16
7B	RCG-3	2.477		711.2	609.6	22.75	365.42	7	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	96.32	9.93	83.57	109.01

TABLE 1

SUMMARY OF FLOOR ELEMENT ANALYSES (continued)

Case	Element Type	L_s (m)	L_L (m)	b_b (mm)	h (mm)	f'_c (MPa)	f_{dy} (MPa)	Support Case	Reinforcing Ratios at Cross Sections 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio Short	Long	Incipient Collapse Overpressure Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)
Building 111, Grant Building (STF)																						
1a	RCS-1	0.466			63.5	13.79	303.37	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	*100.73	4.76	94.67	106.87
1a/r	RCS-1	0.466			63.5	13.79	303.37	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	466.02	47.37	405.34	526.76
2A	RCJ-1	5.486		132.3	317.5	13.79	303.37	7T	0.0237	0.0	--	--	0.0270	0.0205	--	--	0.0135	--	50.13	4.83	43.92	56.26
2B	RCJ-1	2.743		132.3	317.5	13.79	303.37	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	104.32	9.93	91.56	117.07
2C	RCJ-1	1.829		132.3	317.5	13.79	303.37	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	236.35	23.65	206.08	266.62
3A	STG-1	5.537		305.8	408.9		282.69	6					(S=3.68 m^3, 16CI83)						75.64	11.03	61.50	89.77
3B	STG-1	2.769		305.8	408.9		282.69	6					(S=3.68 m^3, 16CI83)						270.62	30.13	232.01	209.23
3C	STG-1	1.846		305.8	408.9		282.69	6					(S=3.68 m^3, 16CI83)						626.25	88.67	512.63	739.95
Building 136, First Federal Savings & Loan Assn. (RCF)																						
1a	RCS-1	0.457	2.083		50.8	20.68	386.11	6	0.0036	0.0	--	--	0.0036	0.0	--	--	0.0036	--	*85.50	8.27	74.88	96.04
1a/r	RCS-1	0.457	2.083		50.8	20.68	386.11	n/a	0.0036	0.0	0.0040	0.0	0.0036	0.0	0.0040	0.0	0.0036	--	488.56	42.20	434.44	542.62
2A	RCJ-1	4.267		157.0	355.6	20.68	365.42	7T	0.0039	0.0039	--	--	0.0043	0.0040	--	--	--	--	24.34	3.38	19.99	28.68
2B	RCJ-1	2.134		157.0	355.6	20.68	365.42	5T	0.0039	0.0039	--	--	--	--	--	--	--	--	66.88	8.48	56.05	77.77
3A	RCG-1	5.791		1067.0	355.6	20.68	365.42	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	29.58	3.86	24.68	34.47
3B	RCG-1	2.896		1067.0	355.6	20.68	365.42	7	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	96.53	11.38	81.91	111.07
3C	RCG-1	1.930		1067.0	355.6	20.68	365.42	5	0.0113	0.0113	--	--	--	--	--	--	--	--	142.72	14.27	124.45	161.06
3H	RCG-1	5.791		1067.0	355.6	20.68	365.42	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	58.61	5.72	51.30	65.98
Building 167, Lafayette Towers Building #2 (RCF)																						
1a	RCFP-1	6.096	6.096		254.0	20.68	365.42	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	43.51	5.24	36.75	50.26
1a/s	RCFP-1	6.096	6.096		254.0	20.68	365.42	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	11.24	0.07	11.17	11.31
1h	RCFP-1	3.048	3.048		254.0	20.68	365.42	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	212.57	27.30	177.54	247.59
1h/s	RCFP-1	3.048	3.048		254.0	20.68	365.42	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	60.47	0.14	60.33	60.60
1i	RCS-1	6.096	6.096		254.0	20.68	365.42	2	0.0030	0.0	0.0030	0.0	0.0030	0.0	0.0030	0.0015	--	--	62.95	6.76	54.26	71.64
1j	RCS-1	3.048	3.048		254.0	20.68	365.42	1	0.0030	0.0	0.0030	0.0	--	--	--	--	--	--	137.21	17.86	114.25	160.10
Building 188, State Wildlife Conservation Building (RCF)																						
1a	RCS-1	4.877	10.262		152.4	20.68	303.37	4	--	--	0.0038	0.0	0.0142	0.0025	--	--	0.0094	--	*72.53	9.65	60.12	84.87
1b	RCS-1	2.057	10.262		152.4	20.68	303.37	1	0.0094	0.0	0.0038	0.0	--	--	--	--	0.0094	--	*143.82	18.68	119.83	167.82
2A	RCB-1	4.013		1219.2	393.7	20.68	303.37	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	51.43	5.24	44.68	58.12
2B	RCB-1	2.007		1219.2	393.7	20.68	303.37	5	0.0086	0.0	--	--	--	--	--	--	0.0051	--	161.48	15.79	141.27	181.75
2H	RCB-1	4.013		1219.2	393.7	20.68	303.37	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	78.19	7.86	68.12	88.25
2I	RCB-1	2.007		1219.2	393.7	20.68	303.37	5	0.0086	0.0	--	--	--	--	--	--	0.0051	--	102.87	10.00	89.90	115.69
3A	RCB-1	5.690		1219.2	393.7	20.68	303.37	7	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0080	--	*30.82	3.86	25.86	35.71
3B	RCB-1	2.845		1219.2	393.7	20.68	303.37	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	106.94	10.86	93.91	119.97
3C	RCB-1	1.897		1219.2	393.7	20.68	303.37	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	168.85	20.20	143.00	194.71

TABLE 1

SUMMARY OF FLOOR ELEMENT ANALYSES (continued)

Case	Element Type	L_s (m)	L_l (m)	b_b (mm)	h (mm)	f'_c (MPa)	f_{dy} (MPa)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure			
									1		2		3		4							
									p	p'	p	p'	p	p'	p	p'	Short	Long	Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)
Building 200, Fitzsimmons General Hospital (RCF)																						
1a	RCS-1	1.981	6.756		101.6	20.68	303.37	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	45.09	3.72	40.33	49.78
1e	RCS-1	1.981	3.378		101.6	20.68	303.37	4	0.0073	0.0	0.0040	0.0	0.0073	0.0073	--	--	--	--	60.47	5.72	53.16	67.84
1h	RCS-1	1.981	2.253		101.6	20.68	303.37	4	0.0073	0.0	0.0040	0.0	0.0073	0.0073	--	--	--	--	71.57	8.07	61.23	81.84
2A	RCB-1	6.756		304.8	457.2	20.68	303.37	6	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	*33.03	3.86	28.06	38.06
2B	RCB-1	3.378		304.8	457.2	20.68	303.37	7	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	95.56	11.51	80.81	110.25
2C	RCB-1	2.252		304.8	457.2	20.68	303.37	5	0.0138	0.0	--	--	--	--	--	--	0.0138	--	119.90	13.86	102.11	137.76
3A	RCG-1	6.477		406.4	711.2	20.68	303.37	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	33.92	3.59	29.30	38.61
3B	RCG-1	3.239		406.4	711.2	20.68	303.37	7	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	117.83	14.82	98.73	136.86
3C	RCG-1	2.159		406.4	711.2	20.68	303.37	5	0.0101	0.0	--	--	--	--	--	--	0.0025	--	196.50	23.86	165.96	227.04
3H	RCG-1	6.477		406.4	711.2	20.68	303.37	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	58.95	6.62	50.40	67.43
4a	RCS-2	2.743	4.572		101.6	20.68	303.37	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	22.61	2.07	19.99	25.23
4e	RCS-2	2.286	2.743		101.6	20.68	303.37	3	0.0040	0.0	0.0073	0.0	--	--	0.0073	0.0073	--	--	41.78	5.03	35.37	48.19
5A	RCB-2	4.496		304.8	406.4	20.68	303.37	6	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	*27.17	4.27	21.72	32.61
5B	RCB-2	2.248		304.8	406.4	20.68	303.37	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	82.67	8.00	72.39	92.87
5E	RCB-2	2.248		304.8	406.4	20.68	303.37	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	128.59	15.38	108.94	148.31
6A	RCB-2	2.667		304.8	508.0	20.68	303.37	6	0.0041	0.0029	--	--	0.0029	0.0041	--	--	0.0070	--	83.91	5.10	77.36	90.46
Building 220, Fidelity Federal Plaza Building (RCF)																						
1a	RCS-1	2.413	7.772		114.3	20.68	365.42	6	0.0043	0.0	--	--	0.0043	0.0021	--	--	0.0043	--	*57.23	7.45	47.71	66.74
1e	RCS-1	2.413	3.886		114.3	20.68	365.42	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	*61.98	6.83	53.23	70.74
1f	RCS-1	2.413	2.591		114.3	20.68	365.42	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	*67.57	6.76	58.95	76.26
2A	RCB-1	7.772		330.2	508.0	20.68	365.42	6	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	*37.92	5.79	30.47	45.37
2B	RCB-1	3.886		330.2	508.0	20.68	365.42	7	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	104.59	12.55	88.53	120.66
2C	RCB-1	2.591		330.2	508.0	20.68	365.42	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	139.34	12.48	123.35	155.27
2D	RCB-1	1.943		330.2	508.0	20.68	365.42	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	251.24	22.41	222.56	279.93
3A	RCG-1	7.925		457.2	914.4	20.68	365.42	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	25.65	2.48	22.48	28.82
3B	RCG-1	3.962		457.2	914.4	20.68	365.42	7	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	84.18	8.62	73.08	95.22
3C	RCG-1	2.642		457.2	914.4	20.68	365.42	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	143.55	14.62	124.86	162.30
3D	RCG-1	1.981		457.2	914.4	20.68	365.42	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	240.21	30.47	201.12	279.31
3H	RCG-1	7.925		457.2	914.4	20.68	365.42	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	54.47	6.48	46.13	62.81
Building 225, Broadway Crenshaw Building (RCF)																						
1a	RCFS-1	7.315	7.315		228.6	17.24	365.42	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	*49.44	6.83	40.68	58.19
1a/s	RCFS-1	7.315	2.235		228.6	17.24	365.42	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	*47.44	5.17	40.82	54.12
1i	RCS-1	7.315	7.315		228.6	17.24	365.42	2	0.0044	0.0	0.0044	0.0	0.0044	0.0	0.0054	0.0	0.0027	0.0027	*60.60	6.83	53.09	70.53
1j	RCS-1	3.658	3.658		228.6	17.24	365.42	1	0.0044	0.0	0.0044	0.0	--	--	--	--	0.0027	0.0027	*125.62	17.65	103.01	148.17
2a	RCS-2	1.905	7.264		101.6	17.24	365.42	5	0.0111	0.0	--	--	--	--	--	--	0.0056	--	*77.84	8.62	66.81	88.87
2e	RCS-2	1.905	3.632		101.6	17.24	365.42	5	0.0111	0.0	--	--	--	--	--	--	0.0056	--	*86.94	10.69	73.29	100.66
2f	RCS-2	1.905	2.421		101.6	17.24	365.42	5	0.0111	0.0	--	--	--	--	--	--	0.0056	--	*89.15	9.51	77.01	101.35
3A	RCB-2	7.264		406.4	635.0	17.24	365.42	7	0.0123	0.0	--	--	0.0126	0.0062	--	--	0.0062	--	51.43	6.48	43.09	59.71
3B	RCB-2	3.632		406.4	635.0	17.24	365.42	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	137.07	13.51	119.83	154.37
3C	RCB-2	2.421		406.4	635.0	17.24	365.42	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	323.64	36.34	277.10	370.25
4A	RCG-2	6.426		660.4	762.0	17.24	365.42	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	85.91	9.86	73.22	98.53
4B	RCG-2	3.213		660.4	762.0	17.24	365.42	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	207.46	21.93	179.40	235.59
4C	RCG-2	2.141		660.4	762.0	17.24	365.42	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	473.12	48.61	410.86	535.45
4H	RCG-2	6.426		660.4	762.0	17.24	365.42	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	174.16	19.99	148.51	199.81

TABLE 1

SUMMARY OF FLOOR ELEMENT ANALYSES (concluded)

Case	Element Type	L_s (m)	L_L (m)	b_b (mm)	h (mm)	f'_c (MPa)	f_{dy} (MPa)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure			
									1		2		3		4							
									p	p'	p	p'	p	p'	p	p'	Short	Long	Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)
Building 227, May Company Eastland Shopping Center (RCF)																						
1a	RCS-1	0.737			76.2	20.68	386.11	6	0.0032	0.0	--	--	0.0032	0.0	--	--	0.0032	--	*70.26	4.55	64.47	76.05
1a/r	RCS-1	0.737			76.2	20.68	386.11	n/a	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	--	355.08	36.20	308.68	401.55
2A	RCJ-1	6.858		190.5	431.8	20.68	365.42	7T	0.0107	0.0	--	--	0.0107	0.0053	--	--	0.0053	--	24.06	2.76	20.55	27.58
2B	RCJ-1	3.429		190.5	431.8	20.68	365.42	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	70.33	9.45	58.19	82.46
2C	RCJ-1	2.286		190.5	431.8	20.68	365.42	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	151.06	17.72	128.38	173.82
3A	RCG-1	6.909		762.0	609.6	20.68	365.42	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	52.19	4.96	45.78	58.54
3B	RCG-1	3.454		762.0	609.6	20.68	365.42	7	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	157.68	15.38	138.03	177.40
3C	RCG-1	2.303		762.0	609.6	20.68	365.42	5	0.0157	0.0	--	--	--	--	--	--	0.0137	--	211.88	22.68	182.78	240.90
3H	RCG-1	6.909		762.0	609.6	20.68	365.42	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	97.70	11.72	82.67	112.80
4A	RCG-1	7.112		762.0	660.4	20.68	365.42	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	46.40	4.48	40.68	52.12
4B	RCG-1	3.556		762.0	660.4	20.68	365.42	7	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	143.89	16.20	123.14	164.65
4C	RCG-1	2.370		762.0	660.4	20.68	365.42	5	0.0118	0.0	--	--	--	--	--	--	--	--	199.26	22.61	170.23	228.22
4H	RCG-1	7.112		762.0	660.4	20.68	365.42	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	98.11	11.24	83.63	112.52
Building 245, Portland Hilton Hotel (RCF)																						
1a	RCS-1	0.457	3.200		76.2	20.68	386.11	6	0.0029	0.0	--	--	0.0029	0.0	--	--	0.0029	--	181.54	1.10	180.09	182.99
1a/r	RCS-1	0.457	3.200		76.2	20.68	386.11	n/a	0.0029	0.0	0.0005	0.0	0.0029	0.0	0.0005	0.0	0.0029	--	1126.74	112.45	982.57	1270.98
2A	RCJ-1	6.553		139.7	381.0	20.68	365.42	6T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	73.43	9.58	61.16	85.70
2B	RCJ-1	3.277		139.7	381.0	20.68	365.42	7T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	216.08	26.61	181.95	250.21
2C	RCJ-1	2.184		139.7	381.0	20.68	365.42	5T	0.0181	0.0103	--	--	--	--	--	--	--	--	273.45	28.54	236.83	310.06
3A	RCG-1	2.299		762.0	1950.7	20.68	365.42	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	67.43	9.45	55.36	79.50
3B	RCG-1	11.494		762.0	1950.7	20.68	365.42	7	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	217.39	27.72	181.81	252.90
3C	RCG-1	7.662		762.0	1950.7	20.68	365.42	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	316.40	31.16	276.41	356.39
3D	RCG-1	5.747		762.0	1950.7	20.68	365.42	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	564.54	54.54	494.63	634.46
3H	RCG-1	2.299		762.0	1950.7	20.68	365.42	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	127.83	17.31	105.63	150.10
3J	RCG-1	2.299		762.0	1950.7	20.68	365.42	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	171.68	18.41	148.10	195.33

TABLE 2

FLOOR ELEMENT TYPE AND SUPPORT KEY

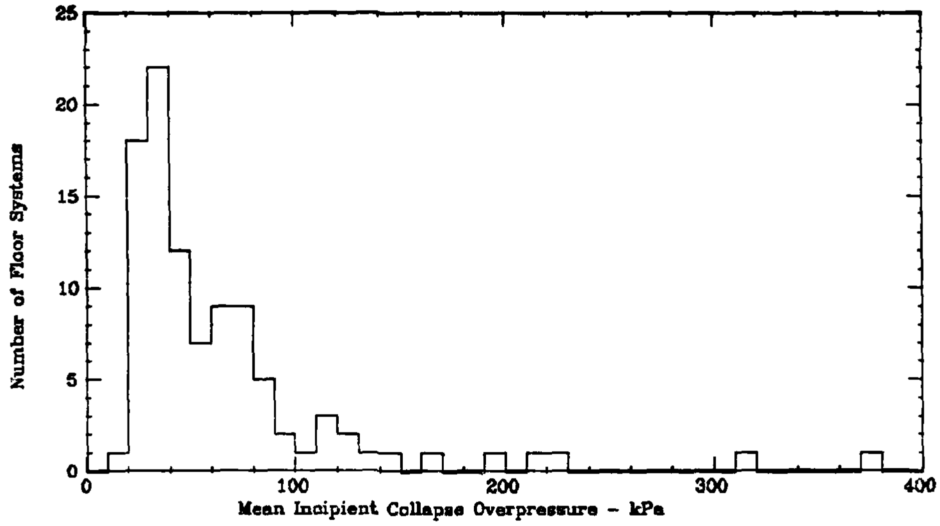
Letter	Floor Element Type
WJ	Wood joist floor
RCFP	Reinforced concrete flat plate
RCFS	Reinforced concrete flat slab (flat plate with drop panels)
RCSB	Reinforced concrete solid slab supported on steel beams
RCS	Reinforced concrete solid slab
RCB	Reinforced conrete beam
RCG	Reinforced concrete girder supporting beams and slabs
RCJ	Reinforced concrete joist supporting slab (in pan-joist system)
STB	Steel beam
STG	Steel girder (joist)

Number	Support Case
1	Two-way, simply supported on four edges
2	Two-way, fixed on four edges
3	Two-way, fixed on short (or vertical) edges; simply supported on long (or horizontal) edges
4	Two-way, simply supported on short (or vertical) edges; fixed on long (or horizontal) edges
5	One-way, simply supported ends
6	One-way, fixed ends
7	One-way, propped cantilever
11	Flat slab or flat plate
T	Used to denote T-beam action; e.g., 5T

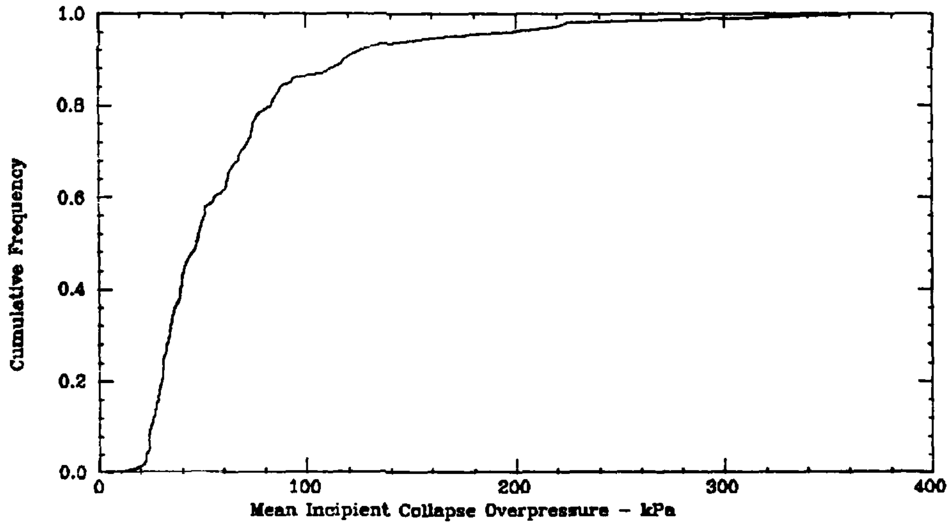
TABLE 3

SUMMARY OF FLOOR SYSTEM ANALYSES

Building	Floor System	Percent of Total Usable Basem't Floor Area	Incipient Collapse Overpressure							
			As Built				Upgraded			
			Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)	Mean (kPa)	Std. Dev. (kPa)	10% Prob. (kPa)	90% Prob. (kPa)
100	1	11	39.23	4.34	33.65	44.75	630.39	59.50	554.13	706.64
	2	2	94.05	11.17	79.77	108.39	116.66	13.58	99.29	134.04
	3	21	40.20	4.28	34.68	45.64	743.88	83.22	637.28	850.54
	4	11	24.20	3.52	19.72	28.75	436.80	51.23	371.09	502.44
	5	20	35.58	4.96	29.23	41.92	347.03	32.27	305.66	388.40
110	1	15	28.61	3.93	23.58	33.65	93.50	9.24	81.64	105.29
	2	24	22.27	4.21	16.96	27.65	46.68	6.48	38.34	54.95
	3	27	28.96	3.31	24.75	33.16	53.71	5.72	46.40	61.09
111	1	56	50.13	4.83	43.92	56.26	236.35	23.65	206.08	266.62
136	1	47	24.34	3.38	19.99	28.68	66.88	8.48	56.05	77.77
167	1	86	11.24	0.07	11.17	11.31	137.21	17.86	114.25	160.10
188	1	40	30.82	3.86	25.86	35.71	143.82	18.68	119.83	167.82
200	1	18	33.03	3.86	28.06	38.06	71.57	8.07	61.23	81.84
	2	27	22.61	2.07	19.99	25.23	41.78	5.03	35.37	48.19
220	1	30	25.65	2.48	22.48	28.82	67.57	6.76	58.95	76.26
225	1	77	49.44	6.83	40.68	58.19	125.62	17.65	103.01	148.17
	2	22	51.43	6.48	43.09	59.71	89.15	9.51	77.01	101.35
227	1	64	24.06	2.76	20.55	27.58	151.06	17.72	128.38	173.82
245	1	27	67.43	9.45	55.36	79.50	273.45	28.54	236.83	310.06

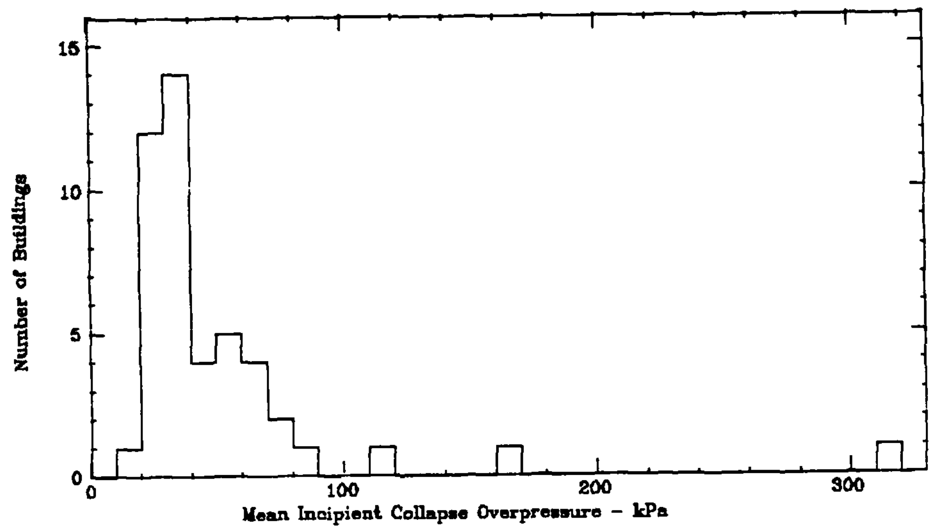


Histogram of Floor System MICOs

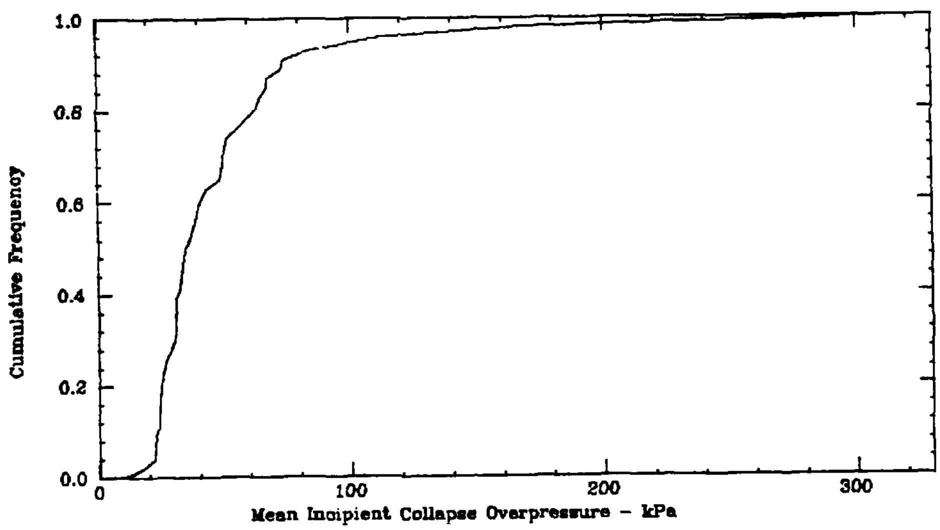


Cumulative Frequency of Floor System MICOs

Figure 1: MEAN INCIPIENT COLLAPSE OVERPRESSURES OF 99 FLOOR SYSTEMS - Histogram and Cumulative Frequency Distribution



Histogram of Building MICOs



Cumulative Frequency of Building MICOs

Figure 2: MEAN INCIPIENT COLLAPSE OVERPRESSURES OF 46 BUILDINGS - Histogram and Frequency Distribution

2.2 EFFECT OF TYPE OF SUPPORT BEAM ON FLOOR SYSTEM STRENGTH

An examination of the analyses of slabs and support beams in slab-and-beam floor systems in Wiehle (1974) revealed that there was a relationship between the relative blast strength of the elements of such a floor system and the type of support beam. For the 36 buildings summarized in Wiehle (1974), there were 47 floor systems in 20 buildings of slab-and-beam (or slab-beam-and-girder) construction. Wiehle found that for slab-and-beam floor systems with steel support beams (32 systems in 12 buildings), the slab would collapse before the beam in 63 percent of the cases. The floor collapse, therefore, was determined by the slab. However, for the floor systems with R/C support beams (15 systems in 8 buildings), the concrete beams would collapse before the slabs in 87 percent of the cases. The data in Wiehle (1974) for the 20 buildings are summarized as follows:

Type of Floor System	System Collapse Controlled by Slab	
	Slab	Beam
Reinforced concrete slab with steel support beams/girders (12 buildings)	20 (63%)	12 (37%)
Reinforced concrete slab with reinforced concrete support beams/girders (8 buildings)	2 (13%)	13 (87%)

Of the 19 floor systems in the present study, 9 systems in 6 buildings were of slab-and-beam construction. The results of the floor analyses for all 26 buildings are summarized:

Type of Floor System	System Collapse Controlled by	
	Slab	Beam
Reinforced concrete slab with steel support beams/girders (12 buildings)	20 (63%)	12 (37%)
Reinforced concrete slab with reinforced concrete support beams/girders (14 buildings)	5 (21%)	19 (79%)

For the total sample of buildings it was found that for the floor systems with steel support beams (32 systems in 12 buildings), the slab would collapse before the steel beams in 63 percent of the cases; and for floor systems with reinforced concrete beams (24 systems in 14 buildings), the beams would collapse before the slab in 79 percent of the cases. These findings are consistent with those reported in Wiehle (1974). The effect of the type of support beam on the MICO of floor systems is shown graphically in Figure 3. This figure shows that systems consisting of R/C slabs supported by steel beams have a substantially greater strength (approximately 70 percent higher at the 50 percent value) than do systems consisting of R/C slabs supported by R/C beams. It can be concluded from this that slabs supported by steel beams generally are better candidates for "as built" shelters because the floor system will develop the full strength of the slab and not be limited by the resistance of the supporting beam(s).

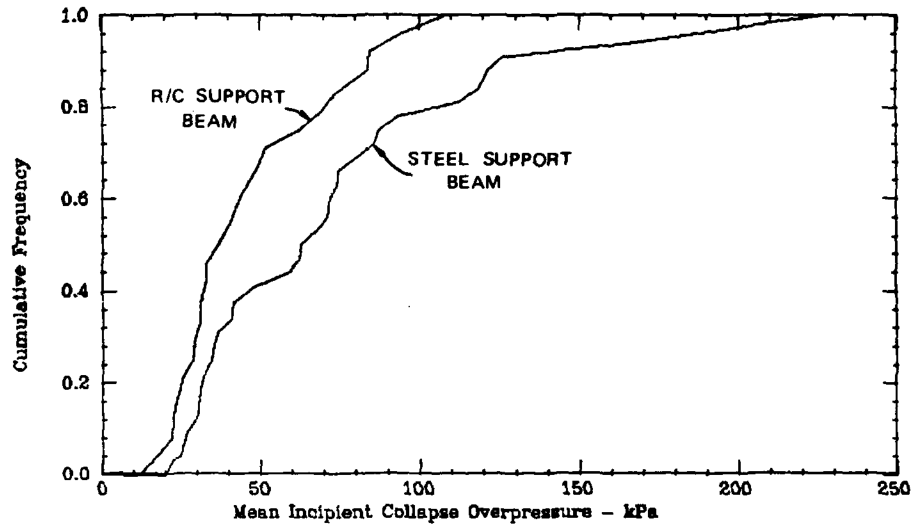


Figure 3: INCIPIENT COLLAPSE OVERPRESSURE OF FLOOR SYSTEMS BY TYPE OF SLAB SUPPORT - Comparison of the Cumulative Frequency Distributions

2.3 CONCLUSIONS

Results of the analysis of the "as built" cases show that floor systems in floor-over-basement areas have a mean incipient collapse overpressure (MICO) of about 50 kPa (≈7 psi). However, the collapse of a floor system represents a localized failure within the entire floor-over-basement area. Adjacent systems may have resistances of 300 kPa (≈45 psi) or more. A building, therefore, can provide open shelter even with the collapse of part of the floor area.

Tensile membrane behavior is of particular importance in predicting the incipient collapse overpressure of flat plate or flat slab systems. Continuity and proper tensile membrane steel anchorage is necessary to allow the full strength of the system to develop and to prevent premature shear failure. However, based on this study and previous work, tensile membrane steel details in most existing flat plate or flat slab systems are not sufficient to assure membrane behavior. Therefore, flat plate or flat slab systems should be used only when other shelter is not available, and they should be analyzed as if there is no tensile membrane steel present, unless there is information to the contrary.

The analysis of the effect of support beam type on slab-and-beam floor system strength gives further support to the tentative conclusion in Wiehle (1974), that, "on the average, floor systems in steel-frame buildings can be expected to be stronger (by as much as a factor of two for some cases) in resisting nuclear air blast than floors in reinforced concrete frames."

Chapter 3

BUILDING ANALYSIS USING UPGRADING SCENARIOS

In this section the analyses of various upgrading scenarios for the floor elements of the 11 buildings in the present effort are discussed. The results of the analyses are summarized in Table 1 along with the "as built" results.

3.1 ANALYSIS

This section consists of three parts paralleling the analysis section of the previous chapter. These parts are floor element analyses, floor system analyses, and building analyses.

3.1.1 Floor Element Analyses

Upgrading was only considered in those cases where the resultant span(s) were greater than 1.8 m (≈6 ft) and the original steel details were compatable with upgrading. The minimum of 1.8 m was chosen because it was the smallest distance that would still allow functional use of the basement area during the construction of the support system and the crisis period. When upgrading was not feasible, the upgraded strength was assumed equal to the "as built" strength. Tables 4 and 5 give the upgrading schemes considered for slabs and beams, respectively. Using Building 100 for illustration, slab 1 was analyzed four ways: as built, supports added at mid-span, supports added at third-span, and supports added at quarter-span. No effort was made to put supports at optimal positions for upgrading. The supports were simply put at mid-, third-, and quarter-span.

For example, if a slab were upgraded with supports at third-points, the center third slab would be fully restrained and therefore have more resistance than the outer third-spans. For the upgraded outer third-spans to have the same strength as the fully restrained center third-span, the support beams would need to be placed closer to the slab ends.

Furthermore, the maximum upgrading potential of beams or girders in every floor system was not calculated since in all systems the upgraded strength of the slab or pan-joist system (supported by the beam or girder) determined the strength of the floor system. With combinations

of multiple supports under slabs (or pan-joist systems) and under beams (or girders), the beams/girders can be made to sustain very large loads providing the columns, foundations, and walls are sufficient.

As can be found in Table 1, the maximum mean upgraded resistance was 1297.1 kPa (188.1 psi) for an individual member, 743.8 kPa (107.9 psi) for a floor system, and 273.5 kPa (39.7 psi) for a building. Fifty percent of the floor systems studied were only upgradable to 120 kPa (17.4 psi). The results of the upgrading are summarized in Figures 4, 5, 6, and 7.

Figure 4 is a plot of the ratio of the mean incipient collapse overpressures (MICOs) of the upgraded to "as built" elements versus the MICOs of the "as built" elements for all the scenarios given in Table 1. The letters used in the plot correspond to the upgrading schemes given in Tables 4 and 5. The "r" stands for the restrained slab "as built" cases.

Figure 5 gives a histogram and cumulative frequency distribution of the ratio of the maximum MICO of the upgraded case to the "as built" case for all the floor elements. As the figure shows, the increase in strength ranged between 1 and 18.5. Fifty percent of the floor elements had a potential to increase their strength by a ratio of 3.6; only 10 percent by a ratio of 9.8 or more.

3.1.2 Floor System Analyses

After the maximum upgraded potential for each element in a floor system was calculated, the upgraded potential of the floor system was determined by the lowest value among the individual elements. Table 3 in the previous chapter gives the results of the upgrading analyses of the 19 floor systems in the present study. In all upgrading cases the floor system strength was limited by either the joist strength of a pan-joist system or the slab strength of a slab-and-beam system. Figure 6 gives a histogram of the upgraded potentials of the floor systems considered. These values would apply if upgrading only part of the basement was feasible. The MICOs range between 41.8 kPa and 743.9 kPa (6.1 to 107.9 psi) with 50 percent of the systems failing at 120 kPa (17.4 psi) or less, and 90 percent collapsing at 470 kPa (68.2 psi) or less.

3.1.3 Building Analyses

After the upgraded potential of each floor system was determined, the maximum upgraded potential of the entire basement was found from the lowest upgraded potential of the floor systems. Figure 7 gives a histogram and frequency distribution of the upgraded potentials of the 11 buildings studied. These numbers would apply if upgrading the whole

basement was considered. The MICOs range between 41.8 kPa and 273.5 kPa (6.1 to 39.7 psi) with the median case being 105 kPa (15.2 psi).

TABLE 4

UPGRADING SCHEMES FOR SLABS

In the following figures, the shaded portions represent the area of the slab analyzed. Added support beams or columns are indicated by broken lines.

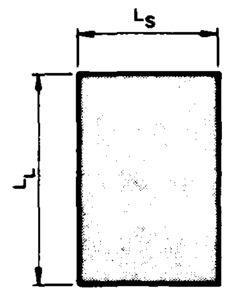
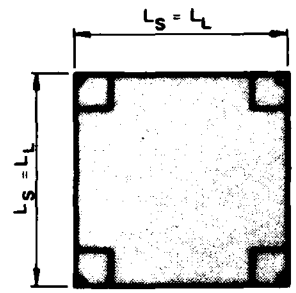
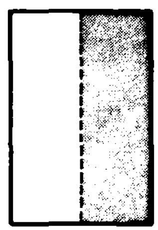
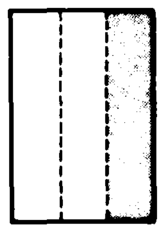
Case	Description	Slab Supported By Beams Figure	Flat Plate/Slab Figure
a	As built		
b	Added support beams parallel to long span		
c	Two added support beams parallel to long span		

TABLE 4

UPGRADING SCHEMES FOR SLABS (continued)

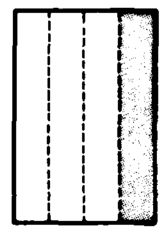
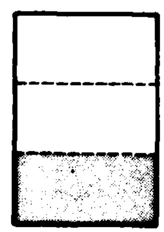
Case	Description	Slab Supported By Beams Figure	Flat Plate/Slab Figure
d	Three added support beams parallel to long span		
e	Added support beam parallel to short span		
f	Two added support beams parallel to short span		

TABLE 4

UPGRADING SCHEMES FOR SLABS (continued)

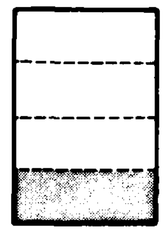
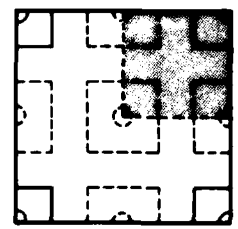
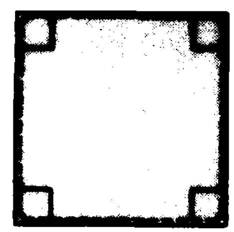
Case	Description	Slab Supported By Beams Figure	Flat Plate/Slab Figure
g	Three added support beams parallel to short span		
h	Added columns at half points		
i	Added support beams around periphery, fixed on all edges		

TABLE 4

UPGRADING SCHEMES FOR SLABS (concluded)

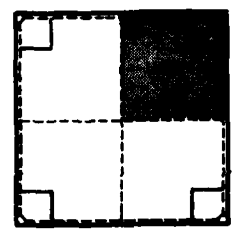
Case	Description	Slab Supported By Beams Figure	Flat Plate/Slab Figure
j	Added support beams at half span in both directions and on all edges as simple supports		

TABLE 5

UPGRADING SCHEMES FOR BEAMS AND GIRDERS

In the following figures, the shaded portions represent the area of the slab supported by the bean or girder analyzed. Added support beams or columns are indicated by broken lines.

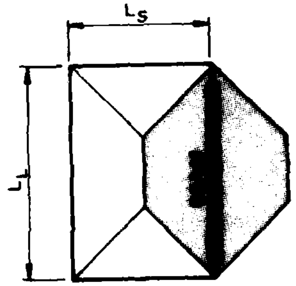
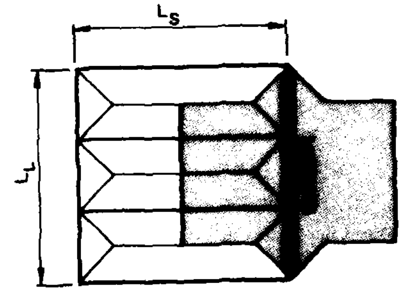
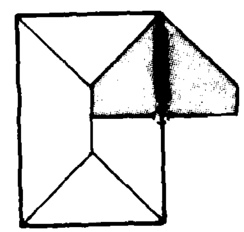
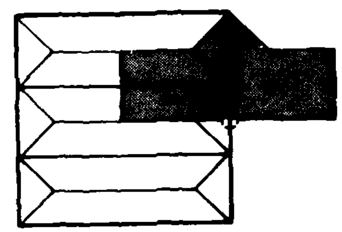
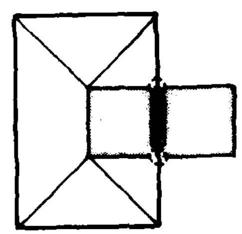
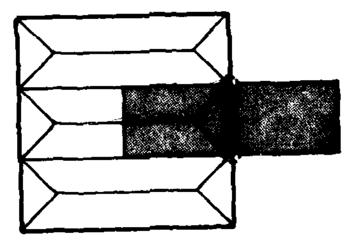
Case	Description	Beam Figure	Girder Figure
A	As built		
B	Beam or girder has added support column at mid-span.		
C	Beam or girder has added support columns at third-span.		

TABLE 5

UPGRADING SCHEMES FOR BEAMS AND GIRDERS (continued)

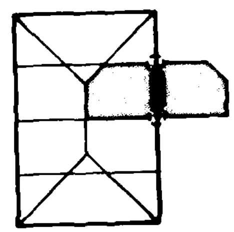
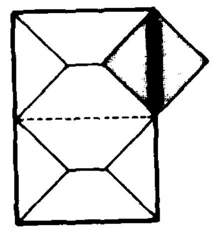
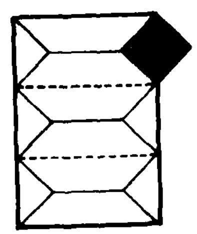
Case	Description	Beam Figure	Girder Figure
D	Beam has added support columns at quarter-span.		
E	Beam and associated slab have added support beam at mid-span.		
F	Beam and associated slab have added support beams at third-spans.		

TABLE 5

UPGRADING SCHEMES FOR BEAMS AND GIRDERS (continued)

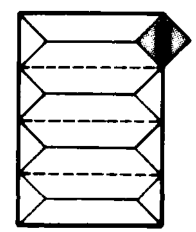
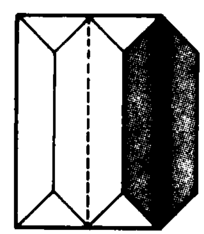
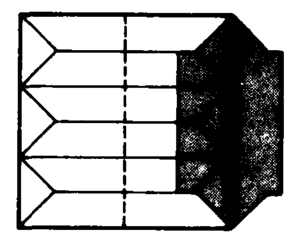
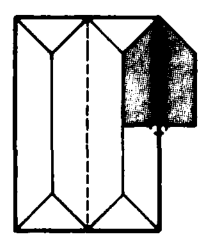
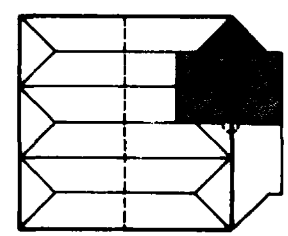
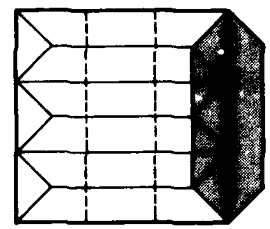
Case	Description	Beam Figure	Girder Figure
G	Beam and associated slab have added support beams at quarter-spans.		
H	Beam case has additional beam supporting slab. Girder case has additional girder supporting beams but not slab.		
I	Beam case has added support column at mid-span and an additional beam supporting the slab. Girder case has added support column at mid-span and an additional girder supporting beams but not slabs.		

TABLE 5

UPGRADING SCHEMES FOR BEAMS AND GIRDERS *(concluded)*

Case	Description	Beam Figure	Girder Figure
J	Girder has two additional girders supporting beams but *not slabs*.		

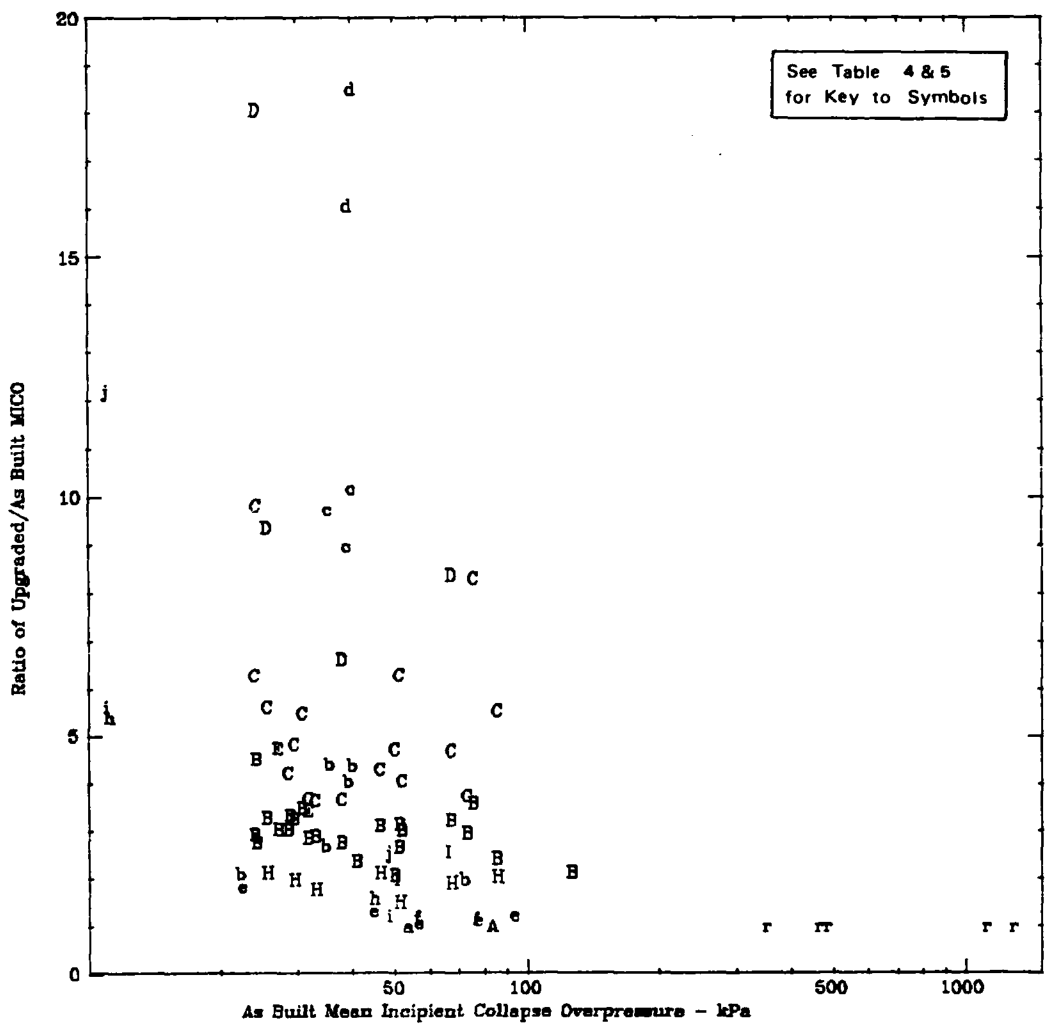
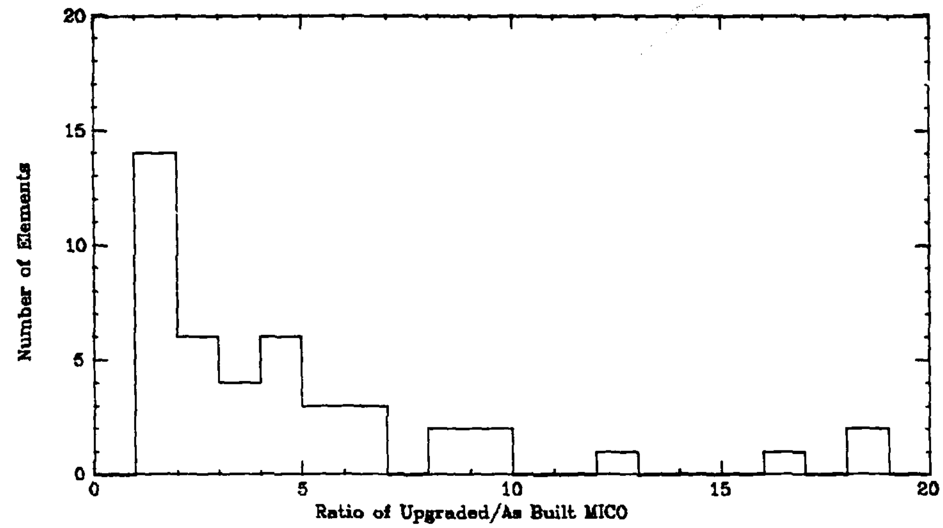
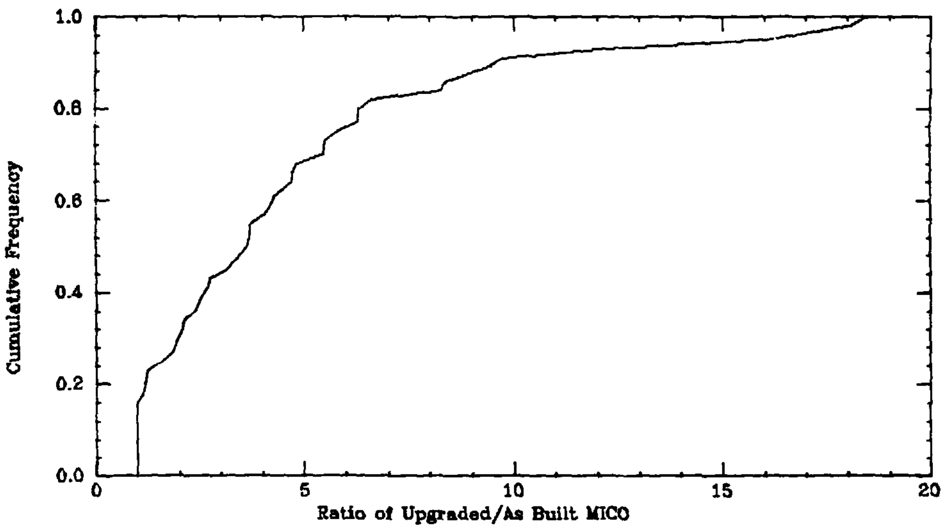


Figure 4: RATIO OF THE MEAN INCIPIENT COLLAPSE OVERPRESSURES OF UPGRADED TO "AS BUILT" CASES VS "AS BUILT" CASES

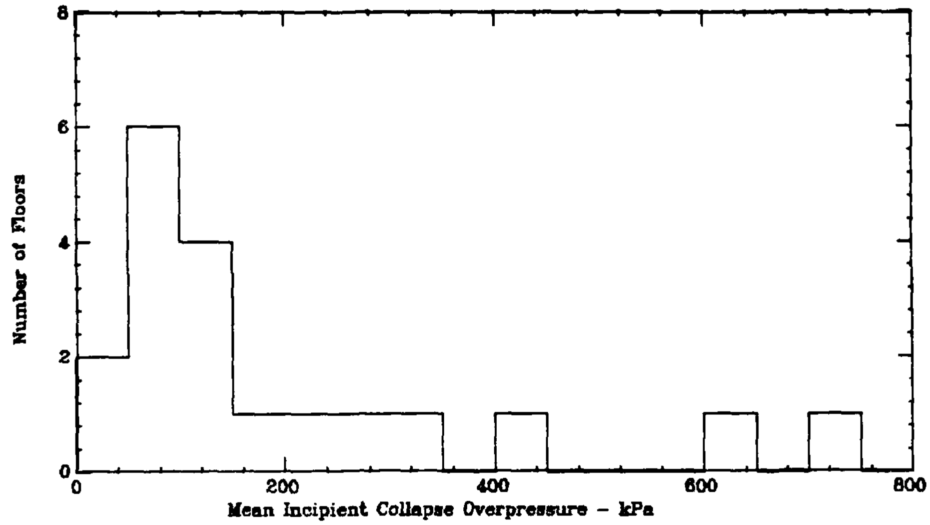


Histogram of Upgraded Element MICOs

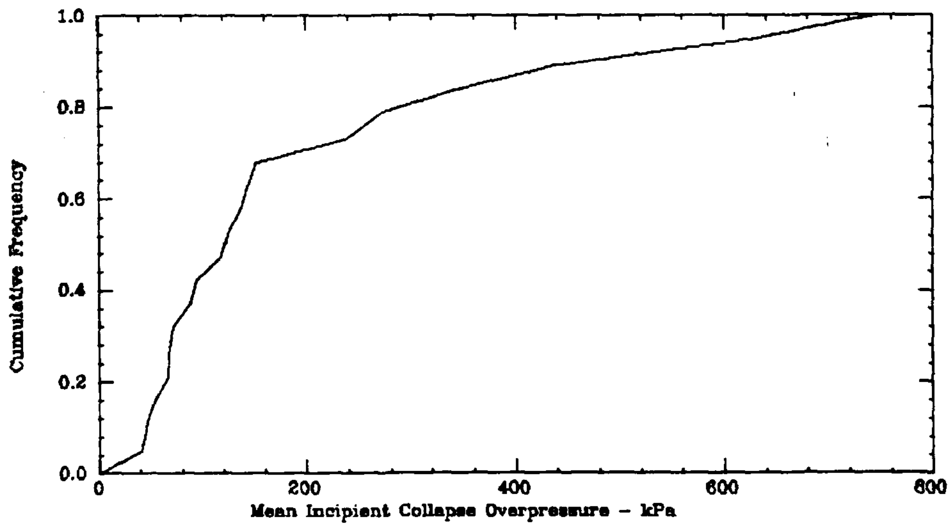


Cumulative Frequency of Upgraded Element Overpressures

Figure 5: RATIO OF THE MEAN INCIPIENT COLLAPSE OVERPRESSURES OF UPGRADED TO "AS BUILT" CASES - Histogram and Cumulative Frequency Distribution

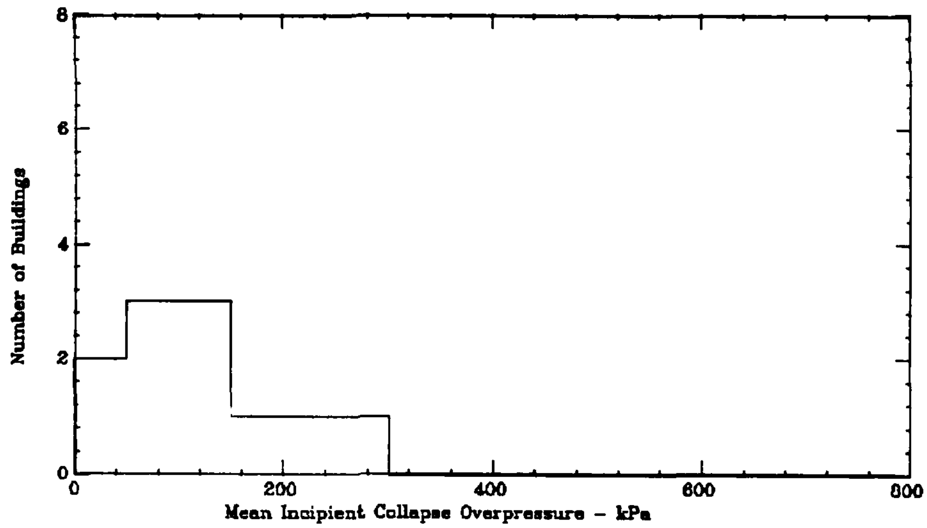


Histogram of Upgraded Floor System Overpressures

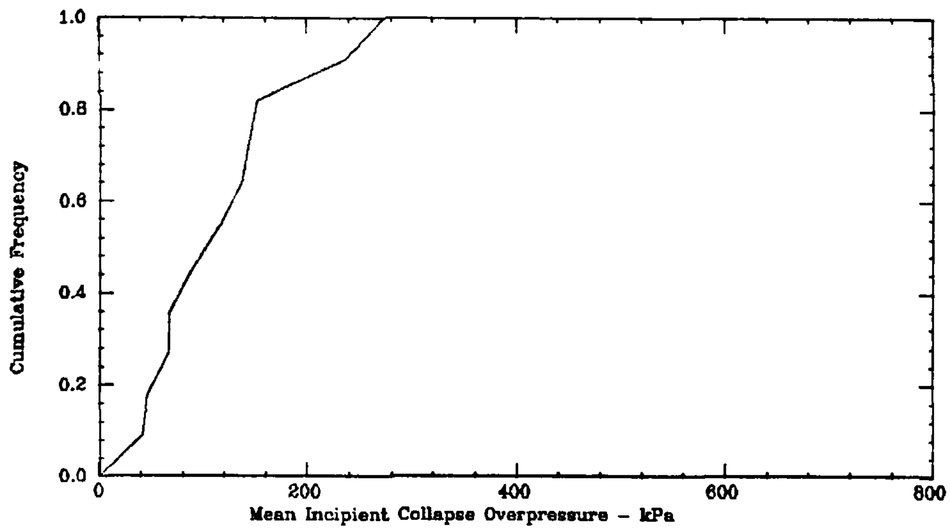


Cumulative Frequency of Upgraded Floor System MICOs

Figure 6: MEAN INCIPIENT COLLAPSE OVERPRESSURES OF UPGRADED FLOOR SYSTEMS - Histogram and Frequency Distribution



Histogram of Upgraded Building MICOs



Cumulative Frequency of Upgraded Building MICOs

Figure 7: MEAN INCIPIENT COLLAPSE OVERPRESSURES OF UPGRADED BUILDINGS - Histogram and Frequency Distribution

3.2 CONCLUSIONS

Using simple floor upgrading schemes (i.e., upgrading by reducing spans alone) would result in overall basement resistance of 41.8 to 273.5 kPa (6.1 to 39.7 psi). The median value of the basements analyzed was 105 kPa (15.2 psi) and two (18 percent) were upgradable to over 210 kPa (≈30 psi).

One simple observation about upgrading potential for the sample evaluated is that the best candidates for upgrading are those buildings that have: (a) R/C slabs or R/C pan-joist systems with large free spans [i.e., greater than 5.5 m (18 ft) as does Building 100] that allow multiple intermediate supports; and (b) their principal steel sufficiently anchored to avoid tensile pullout or shear failure of the member. Further calculations in Appendix C also indicate that slabs 200 mm (≈8 in.) thick are also good candidates for upgrading. This thickness gives the slab a MICO of 350 kPa (≈50 psi) when the span is reduced to 1.8 m by intermediate supports.

It is emphasized that the 11 buildings evaluated and reported on herein were analyzed only for their floor-over-basement area resistance, and not for the resistance of walls, columns or foundations. The upgrading potential of foundations and exterior basement walls (as shown in Appendix D) may be of great importance if shelter is to be provided to withstand overpressures of 206.8 to 344.7 kPa (30 to 50 psi).

Chapter 4

CONCLUSIONS AND RECOMMENDATIONS

4.1 CONCLUSIONS

Conclusions were given at the end of each chapter and in the appendices where appropriate. They are summarized below:

1. "As built" floor-over-basement areas can be expected to show localized failures at overpressures of approximately 50 kPa (≈7 psi) in 50% of the cases studied.

2. Careful study should be made before using a flat plate or flat slab system for shelter due to shear and tensile membrane problems.

3. Floor systems consisting of R/C slabs on steel beams can be expected to be stronger than systems consisting of R/C slabs supported by R/C beams.

4. Floor systems can be upgraded to resist overpressures of greater than 210 kPa (≈30 psi). This study, with an admittedly small sample, shows that about 15 percent of the cases are upgradable to this overpressure.

5. The best floor-over-basement system candidates for upgrading are areas with large free spans. These sections usually contain slabs 150 mm (6 in.) to 200 mm (8 in.) thick that can be upgraded to 210 kPa (≈30 psi) and 350 kPa (≈50 psi), respectively, providing the spans are reduced to 1.8 m (6 ft).

6. The analyses compare very well with experimental behavior. Values observed in the tests were all within ± 15 percent (one standard deviation) of the predicted MICO.

7. Simple calculations and test data show that a 76 mm (3 in.) slab made with standard 19-in. (483 mm) waffle slab pans can be expected to have a MICO of 4900 kPa (710 psi). Slabs made with standard 30-in. (762 mm) waffle slab pans can be expected to reach approximately 1400 kPa (209 psi). The slab portion of a waffle floor system, therefore, presents no problem when considering upgrading to 350 kPa (≈50 psi).

4.2 RECOMMENDATIONS

Some recommendations for further study are listed below:

1. Finish analyzing all NSS buildings in the RTI sample to form a larger representative sample from which better predictions can be made regarding system strengths.

2. Based on the total NSS building sample, develop simplified procedures for predicting MICOs of "as built" basement shelter areas. Existing procedures would be modified to require input of easily observable values such as type of construction, span lengths and thickness. Structural plans would not be required.

3. If upgrading of floor-over-basement slabs is to be pursued, the strength of exterior basement walls and their foundations should be examined through tests (both static and dynamic) and analytical procedures. Buried basement wall data should first be investigated through a literature search to determine the availability of data on exterior walls tested to failure. If no data are available, some testing will be required before the loads on the wall can be predicted accurately enough to determine incipient collapse.

4. Development of a two-degree-of-freedom model to analyze floor systems. This would make a simplified logical progression from the SDOF model to allow the analysis of those cases having a high degree of interaction between the slab and support beams. This interaction could be very significant in cases where the slab has a resistance close to or greater than that of the beam.

5. Make computer programs more user-oriented so that persons unfamiliar with structural dynamics could obtain useful results with minimal work. At present the programs are in a working state and would have to be further simplified so that engineering judgment played less part in their utilization. Limitations on input parameters should be built-in to insure realistic value ranges along with warnings on outputed predictions when they are out of the range of applicable values for the program. Currently all matters of input and output validity are determined by engineering judgment. This recommendation would require refinement of the programs and production of a user's manual.

Appendix A

DETAILED ANALYSIS EXAMPLE (BLDG 100, U.S. POST OFFICE, HARRISBURG, PA)

In this appendix, the procedures followed in analyzing a building are given, ranging from deciding which building is a candidate for analysis through deciding which upgrading scenarios are appropriate to it. To illustrate the steps involved, Building 100, the first building in the NSS series studied herein, is used.

A.1 CHOOSING A BUILDING

The most important criterion used in choosing which NSS buildings to analyze is the availability of a complete set of structural drawings including steel detailing. This, preferably, is augmented by a complete NSS survey form and photographs.

It is also desirable for the building to have multiple stories. Not only do multistory buildings have heavier foundations, exterior walls and columns to support greater loads, they also tend to have greater area per floor level and thus more usable basement shelter area. Adequate basement shelter area is a requirement in the buildings selected.

Once a building is chosen, the basic description of the building, an outline plan of the first floor, and a photograph of the building can be taken from the building plans and accompanying data. For Building 100, the description of the building is as follows:

Description

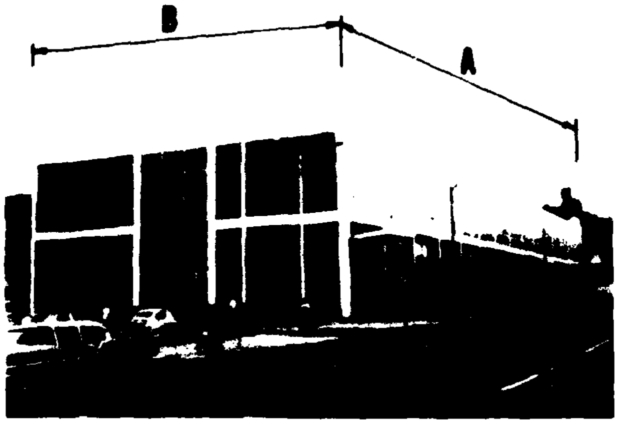
The U.S. Post Office building, constructed in 1960, is located at 813 Market Street, Harrisburg, Pennsylvania. The use class is 49, Government and Public Service, Other, and the building consists of two stories and a partial basement (conveyor tunnels and one large storage room; story height is 2.6 m (8 ft 8 in.) tunnel floor to first floor level). The overall height of the building is about 14 m (46 ft), and plan dimensions of 98.2 m by 203 m (322 ft by 666 ft) provide an area of about 2760 km^2 (29,710 ft^2) on the partial basement level and 10,937 km^2 (117,730 ft^2) on the two aboveground levels. Figure 8 shows the exterior walls and plan of the building at the first floor level. The unanalyzed sections of the

partial basement are shown by dashed lines on the figure. The shaded areas represent the areas of the partial basement that were analyzed.

The building partial basement and first floor are reinforced concrete (R/C) frame construction with nonrigid-frame column and beam connections. Both R/C and structural steel construction is used above the first floor. The floor system (basement and first floor) consists primarily of R/C one-way slabs, beams, columns, and footings [all specified as 20.7 MPa (3,000 psi) concrete at 28 days]. Reinforcing steel was specified as A15 intermediate grade, except column bars and dowels to be hard grade; it was assumed for the analysis that the rebars have a mean dynamic yield strength of 365.4 MPa (53,000 psi), with a standard deviation of 45.7 MPa (6,625 psi).

The exterior walls are constructed of infilled brick areas or large glass areas or a mixture of the two. The basement walls separating excavated and unexcavated areas are constructed of reinforced concrete.

SIDES
A & B



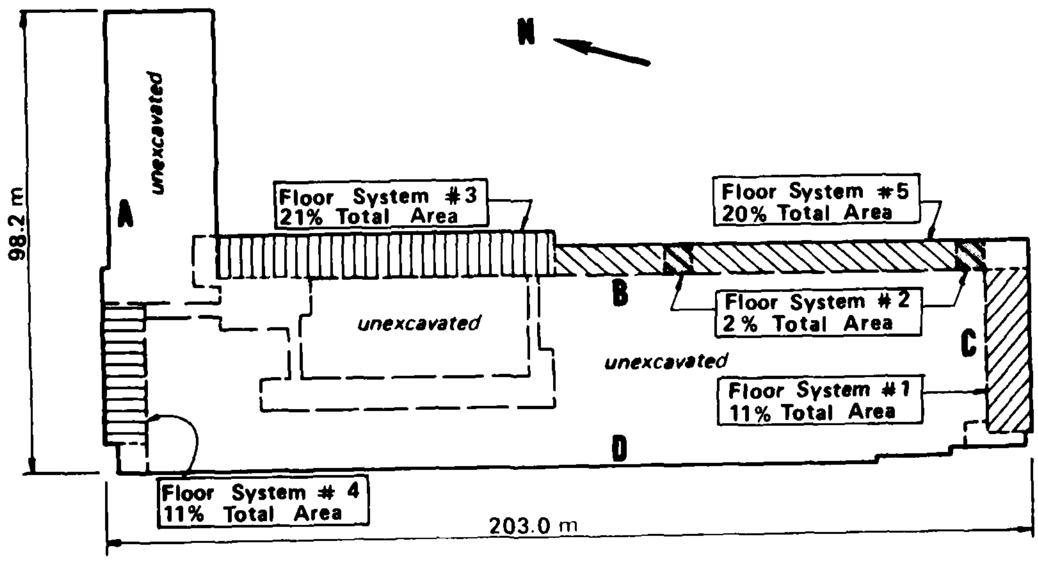


Figure 8: BLDG 100, U.S. POST OFFICE, Harrisburg, PA
- Photograph and Plan of First Floor

A.2 SELECTING FLOOR SYSTEMS

The structural drawings are studied to locate all the various floor systems over the basement shelter area. The systems singled out for analysis are those that are determined to be the weakest based on thickness, amount of reinforcing steel, attachment of the reinforcing steel, or greatest length of clear span of the component elements, or other properties of a floor system that in engineering judgment make it weaker than the other systems over the shelter area. Each selected system is located on the outline drawing, and the percentage it represents of the total usable basement shelter area is shown (see Figure 8).

A.3 "AS BUILT" ANALYSIS

At this point the individual elements of each system are determined and data input forms (developed for use with the SRI computer programs) are prepared to pull together all the information needed in each analysis. Figures 9 through 11 show data input forms prepared for the elements in floor system #4 of Building 100 for the "as built" cases. Figures 9 and 10 give the standard and fully restrained pan-joist slab case; Figure 11 the beam case. (Data input forms for other elements, such as walls, are available but only those used in the example case are shown.) English units are used because the computer programs are currently written for them, and the information on the structural drawings or survey forms is also in English units. In the absence of data on material properties, values based on American Society for Testing and Materials (ASTM) codes of the period are substituted. A summary of the structural properties of materials according to ASTM is given in Table 6. The support case is determined by engineering judgment (see Table 2 for key to support cases).

The need for information on reinforcing steel at different sections is based on the support case chosen. Whether there is adequate embedment of the steel to allow development of the tensile membrane mode is again a matter of engineering judgment.

(The two boxes referring to Load Data and Room-Filling Data are not applicable to this study.)

Computer analyses are made to determine the "as built" strength of the various elements. The results along with the physical properties are entered into Table 1, and a short discussion of the results is written up in a section entitled "Floor Systems" for each building. For Building 100, these are the results:

Floor Systems

In Building 100, five separate floor systems were selected for analysis. The location of each system and the per-

DATA INPUT FORM -- REINFORCED CONCRETE SLAB

Building 100 Case 5a

Location of Slab SLAB OF PAN - JOIST

Slab Data Support Case = RC - 6

L_s = ___ ft ___ in. (= 17.67 in.)

L_t = ___ ft ___ in. (= 82.5 in.)

h_t = 3 in.

f'_c = 3000 psi

f_{d_y} = 53000 psi (mean); 6625 psi (standard deviation)

Section	A_s (sq in./ft)	d (in.)	A'_s (sq in./ft)	d' (in.)
1	0.1305	1.5	0	0
2				
3	0.1305	1.5	0	0
4				

(If tensile membrane to be included) Reinforcement continuous in:

short direction = 0.1305 sq in./ft; long direction = 0 sq in./ft

Comments ______

~~Load Data (Load Type 1)~~

~~W = 1,000 kt P_o = 14.7 psi c_o = 1120 fps~~

~~S (mean) = ___ ft S_{min} = ___ ft~~

~~S (std. dev.) = ___ ft S_{max} = ___ ft~~

~~Room-Filling Data No. of Openings = ___ Air Density = 0.076 pcf~~

~~V_R = ___ ft x ___ ft x ___ ft = ___ cu ft~~

~~A_{win} (sq ft) = ___ ___ ___~~

~~Location Code = ___ ___ ___~~

~~Delay (msec) = ___ ___ ___~~

~~Delay to Initial Loading of Interior Wall ___ msec~~

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
41.55	4.49	35.80	47.30

Figure 9: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 5a

LONGITUDINALLY RESTRAINED
DATA INPUT FORM -- REINFORCED CONCRETE SLAB

Building 100 Case 5 a/r

Location of Slab · SLAB OF PAN - JOIST

Slab Data

Support Case = RC - 2

L_s = ___ ft ___ in. (=17.67 in.) DIF = ___ %

L_L = ___ ft ___ in. (=82.5 in.)

h_t = 3.0 in.

f'_c = 3000 psi (mean); 450 psi (standard deviation)

f_y = 44167 psi (mean); 6625 psi (standard deviation)

Section	A_s (sq in./ft)	d (in.)	A'_s (sq in./ft)	d' (in.)
1	0.1305	1.5	0	0
2	0.021	1.5	0	0
3	0.1305	1.5	0	0
4	0.021	1.5	0	0

Reinforcement continuous in:
short direction = 0.135 sq in./ft; long direction = 0 sq in./ft

Longitudinal edge displacement (per unit length) in:
short direction = 0 in./in.; long direction = 0 in./in.

Load Data (Load Type 1)

W = 1,000 kt P_o = 14.7 psi c_o = 1120 fps

S (mean) = ___ ft S_{min} = ___ ft

S (std. dev.) = ___ ft S_{max} = ___ ft

Room-Filling Data No. of Openings = ___ Air Density = 0.076 pcf

V_o = ___ ft x ___ ft x ___ ft = ___ cu ft

A_{win} (sq ft) = ___ ___ ___

Location Code = ___ ___ ___

Delay (msec) = ___ ___ ___

Delay to Initial Loading of Interior Wall ___ msec

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
188.13	18.95	168.83	212.42

Figure 10: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 5a/r

DATA INPUT FORM -- REINFORCED CONCRETE SUPPORT BEAM

Building 100 Case 6A
Location ______ Comments ______

Loaded Area Data Number of Slabs Supported by Beam = 2

Slab No. 1: Contributary Area = 22.79 sq ft; Thickness = 3 in.
Slab No. 2: Contributary Area = 22.79 sq ft; Thickness = 3 in.

Reinforced Concrete Beam Data

L_b = 30 ft ___ in. (= 360 in.)
b_b = 7.28 in.
h_b = 17 in.
f'_c = 3000 psi
f_{dy} = 53000 (mean) psi; 6625 (std.dev.) psi

Tee Beam Data (Optional)

Beam Spacing = 26 in.
h_s = 3 in.
A_s (slab) = .02145 sq in./ft
d (slab) = 1.34 in.

Support Case = RC - 5

Section	A_s (sq in.)	d (in.)	A'_s (sq in.)	d' (in.)
1	2.27	15.36	0	0
3				

Continuous tensile membrane reinforcement = 0 sq in. (Optional)

Load Data W = 1,000 kt P_o = 14.7 psi c_o = 1120 fps

S (mean) = ______ ft S_{min} = ______ ft
S (std. dev.) = ______ ft S_{max} = ______ ft

Room-Filling Data No. of Openings = ____ Air Density = 0.076 pcf

V_R = ____ ft x ____ ft x ____ ft = ______ cu ft
A_{eff} (sq ft) = ______ ______ ______
Location Code = ______ ______ ______
Delay (msec) = ______ ______ ______

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
3.51	0.51	2.86	4.17

Figure 11: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 6A

TABLE 6

STRUCTURAL PROPERTIES OF MATERIALS (ASTM)

ASTM #	Tensile Yield (kPa)	Years in Effect	Design Allowable Tension (kPa)	Dynamic Tensile Yield (kPa)	Standard Deviation (kPa)
Structural Steel					
A-9	206.8	1900-1924	--	275.8	34.5
A-9	213.7	1924-1936	--	282.7	35.3
A-9	227.5	1936-1939	--	303.4	37.9
A-7	227.5	1939-1972	137.9	303.4	37.9
A-36	248.2	1961-	151.7	331.0	41.4
Reinforcing Steel Bars					
A-15 (Billet)					
Struct.	227.5	1900-1968	124.1	303.4	37.9
Inter.	275.8	1900-1968	137.9	365.4	45.7
Hard	344.7	1900-1968	137.9	365.4	45.7
A-432 (New Billet)					
		1959-1968	165.5	455.1	56.9
A-615 (Billet Deformed)					
Struct.	275.8	1968-	165.5	358.5	44.8
Inter.	413.7	1968-	165.5	455.1	56.9
Hard	517.1	1968-	165.5	455.1	56.9
Reinforcing Wire Mesh					
A-82(cold-drawn)		1921-1968	137.9	386.1	4.3
A-82		1968-	165.5	386.1	4.3 <= 10GA
		1968-	165.5	448.2	56.0 >= 11GA

Concrete

f'_c = 25.9 MPa (3750 psi); standard deviation = 3.2 MPa (470 psi)

centage of total floor-over-usable-basement area it represents are shown in Figure 8. Figure 12 diagrams floor systems #2 and #4. Because of their structural simplicity, systems #1, #3, and #5 were not illustrated.

Systems #1, #3, and #5 involve slabs with very large clear spans. In fact, the large slabs spanning from wall-to-wall comprise the entire system. Results of the analysis of the "as built" case range between 35 and 40 kPa (5 and 6 psi) MICO for the slabs in these three systems.

System #2 is a slab and beam construction. The slab portion has a strength of 94 kPa (13.6 psi). The beam portion of system #2 has a predicted MICO of 126.5 kPa (18.4 psi).

The #4 floor system is a pan-joist system, i.e., a system of closely spaced "T" beams supporting long narrow slabs. The MICO prediction for the slab portion of this system is 286.5 kPa (41.5 psi) using the standard method of analysis. However, because of the nature of the pan-joist system, the slab was also analyzed as being fully restrained. This gave a MICO of 1297.1 kPa (188.1 psi). This second prediction is felt to be closer to the real strength of the member. The beam portion of the system was analyzed as 24.2 kPa (3.5 psi).

For floor system #1, the MICO was predicted at 39.2 kPa (5.7 psi); for system #2, 94.1 kPa (13.6 psi); for system #3, 40.2 kPa (5.8 psi); for system #4, 24.2 kPa (3.5 psi); and for system #5, 35.58 kPa (5.2 psi).

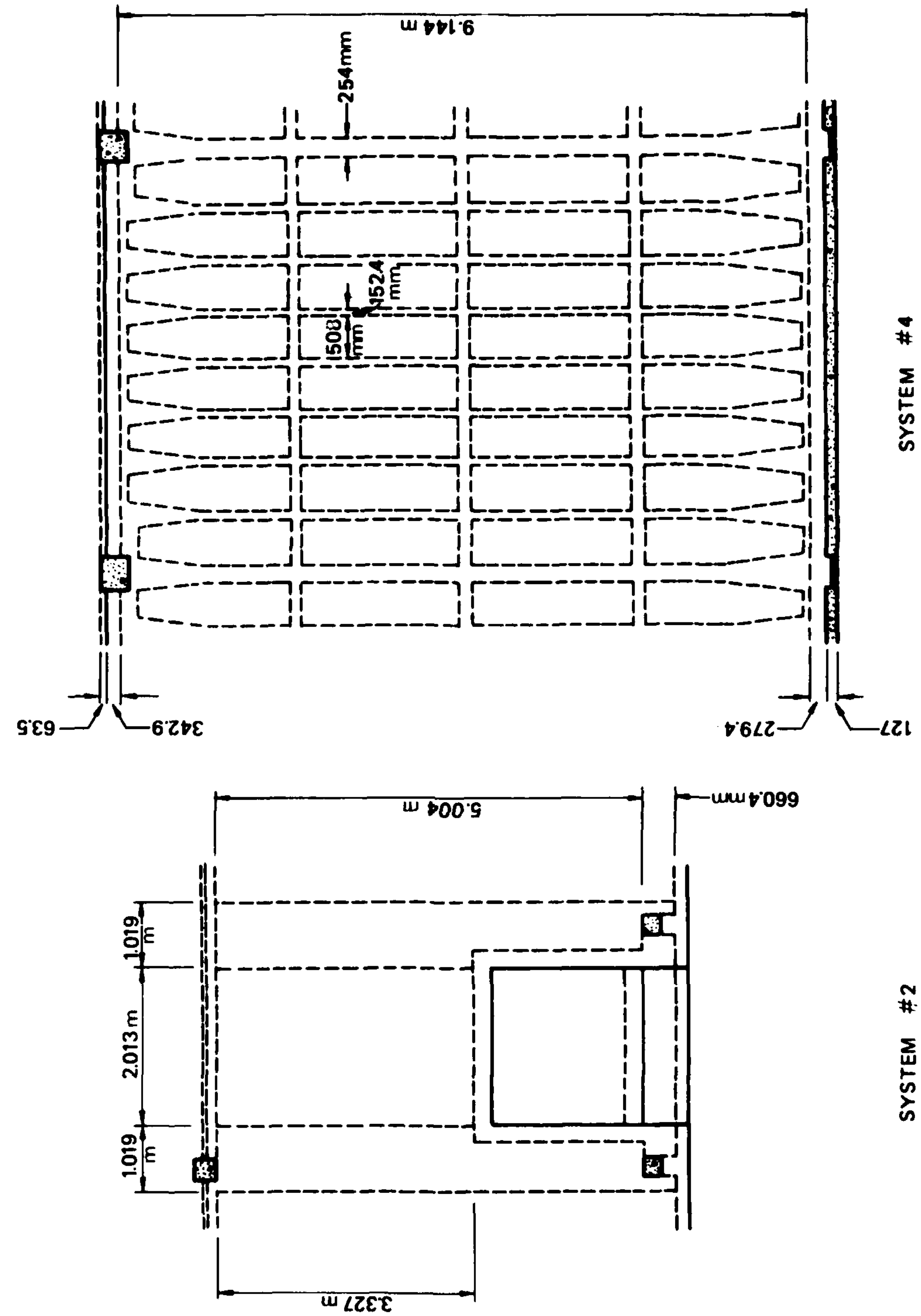


Figure 12: BLDG 100, U.S. POST OFFICE - Floor Systems

A.4 ANALYSIS USING UPGRADING SCENARIOS

The individual elements are now studied to determine if they meet the two requirements set in this study for upgradability--adequate steel detailing and a span, after upgrade reduction, of 1.83 m (6ft) or greater. The upgrading schemes presented in this report are very simple; there is no effort to optimize each case. Spans are reduced through mutliple even divisions. Although each element is treated separately, a check is made that combinations of scenarios given will be structurally consistent within the floor system. If a slab can be reduced with added support beams parallel to the L_L dimension, this scenario is chosen over one calling for a reduction in the L_L dimension because of the greater increased strength. In some cases where the slab controls the upgrading potential and the resultant MICO is low, not all the beam scenarios possible are pursued.

According to the upgrading schemes chosen, changes to the input data for the computer analysis are made. Figures 13 through 15 give examples of data forms prepared for the upgrading cases. These three figures show upgrading cases for the beam in floor system #4; no upgrading scheme was developed for slabs in pan-joist systems. Changes usually involved a reduction in the L_L or L_s dimension, a reduction in the area supported by a beam, and/or a change in support case. The results of the analysis runs on the upgraded cases are then entered into Table 1. A brief discussion of the upgradability of the floor systems and the building is presented under "Upgrading Potential" for each building. For Building 100, this section is shown below.

Upgrading Potential

Floor systems #1, #3, and #5 are unusual among the current sample of NSS buildings because of their long clear slab spans. They offer excellent upgrading potential in two ways:

1. They were strongly built initially to withstand the stresses inherent with long spans, and

2. The long spans allow for additional supports at third- and quarter-span intervals.

Consequently upgrading scenarios produce MICOs of 347.0 kPa (50.3 psi) or more for added third-span supports and up to 743.9 kPa (107.9 psi) for added quarter-span supports.

For system #2, the slab was upgradable by a single added beam parallel to the short span. The increased strength was 116.7 kPa (16.9 psi). The beam in this system also allowed only one upgrading scheme--one added support column at mid-span for a MICO of 269.0 kPa (39.0 psi).

For the pan-joist system, the beam portion was able to reach 436.8 kPa (63.4 psi) based on added quarter-span sup-

ports. No logical procedures were found for upgrading the slab portion; however, the initial strength of the slab effectively eliminated this as a problem. The upgraded slab strength was the same as the "as built" strength, 1297.1 kPa (188.1 psi).

The upgrading potential for this building is determined by the beam in system #4 at 436.8 kPa (63.4 psi).

DATA INPUT FORM -- REINFORCED CONCRETE SUPPORT BEAM

Building 100 Case 6B

Location ____ Comments ____

Loaded Area Data Number of Slabs Supported by Beam = ____

Slab No. 1: Contributary Area = 11.40 sq ft; Thickness = 3 in.

Slab No. 2: Contributary Area = 11.40 sq ft; Thickness = 3 in.

Reinforced Concrete Beam Data

L_b = ___ ft ___ in. (= 180 in.)

b_b = 7.28 in.

h_b = 17 in.

f'_c = 3000 psi

f_{dy} = 53000 psi (mean); 6625 psi (std.dev.)

Support Case = RC - ____

Section	A_s (sq in.)	d (in.)	A'_s (sq in.)	d' (in.)
1	2.27	15.36	0	0
3				

Continuous tensile membrane reinforcement = 0 sq in. (Optional)

Tee Beam Data (Optional)

Beam Spacing = 26 in.

h_s = 3 in.

A_s (slab) = .02145 sq in./ft

d (slab) = 1.34 in.

Load Data W = 1,000 kt P_s = 14.7 psi c_s = 1120 fps

S (mean) = ____ ft S_{sw} = ____ ft

S (std. dev.) = ____ ft S_{ell} = ____ ft

Room-Filling Data No. of Openings = ____ Air Density = 0.076 pcf

V_s = ____ ft x ____ ft x ____ ft = ____ cu ft

A_{eff} (sq ft) = ____ ____ ____

Location Code = ____ ____ ____

Delay (msec) = ____ ____ ____

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
15.82	1.98	13.28	18.35

Figure 13: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 6B

DATA INPUT FORM -- REINFORCED CONCRETE SUPPORT BEAM

Building 100 Case 6C

Location ______ Comments ______

Loaded Area Data Number of Slabs Supported by Beam = 2

Slab No. 1: Contributary Area = 7.8 sq ft; Thickness = 3 in.

Slab No. 2: Contributary Area = 7.8 sq ft; Thickness = 3 in.

Reinforced Concrete Beam Data

L_b = ___ ft ___ in. (= 120 in.)

b_b = 7.28 in.

h_b = 17 in.

f'_c = 3000 psi

f_{dy} = 53000 psi; 6625 psi (mean) (std.dev.)

Support Case = RC - 5

Section	A_s (sq in.)	d (in.)	A'_s (sq in.)	d' (in.)
1	2.27	15.36	0	0
3				

Continuous tensile membrane reinforcement = 0 sq in. (Optional)

Tee Beam Data (Optional)

Beam Spacing = 26 in.

h_s = 3 in.

A_s (slab) = .02145 sq in./ft

d (slab) = 1.34 in.

Load Data W = 1,000 kt P_o = 14.7 psi c_o = 1120 fps

S (mean) = ______ ft S_{an} = ______ ft

S (std. dev.) = ______ ft S_{a11} = ______ ft

Room-Filling Data No. of Openings = ____ Air Density = 0.076 pcf

V_R = ____ ft x ____ ft x ____ ft = ______ cu ft

A_{w11} (sq ft) = ______ ______ ______

Location Code = ______ ______ ______

Delay (msec) = ______ ______ ______

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
34.47	3.3	30.25	38.70

Figure 14: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 6C

DATA INPUT FORM -- REINFORCED CONCRETE SUPPORT BEAM

Building 100 Case 6D

Location ______ Comments ______

Loaded Area Data Number of Slabs Supported by Beam = 2

Slab No. 1: Contributary Area = 5.85 sq ft; Thickness = 3 in.

Slab No. 2: Contributary Area = 5.85 sq ft; Thickness = 3 in.

Reinforced Concrete Beam Data

L_b = ___ ft ___ in. (= 90 in.)

b_b = 7.28 in.

h_b = 17 in.

f'_c = 3000 psi

f_{dy} = 53000 psi; 6625 psi
(mean) (std.dev.)

Tee Beam Data (Optional)

Beam Spacing = 26 in.

h_s = 3 in.

A_s (slab) = .02145 sq in./ft

d (slab) = 1.34 in.

Support Case = RC - 5

Section	A_s (sq in.)	d (in.)	A'_s (sq in.)	d' (in.)
1	2.27	15.36	0	0
3				

Continuous tensile membrane reinforcement = 0 sq in. (Optional)

Load Data W = 1,000 kt P_o = 14.7 psi c_o = 1120 fps

S (mean) = ______ ft S_{sn} = ______ ft

S (std. dev.) = ______ ft S_{sl} = ______ ft

Room-Filling Data No. of Openings = ____ Air Density = 0.076 pcf

V_R = ____ ft x ____ ft x ____ ft = ______ cu ft

A_{wi} (sq ft) = ______ ______ ______

Location Code = ______ ______ ______

Delay (msec) = ______ ______ ______

Predicted Collapse Overpressure, psi

Mean	Standard Deviation	10% Probability	90% Probability
63.35	7.43	53.82	72.87

Figure 15: DATA INPUT FORM, BLDG 100, SYSTEM #4, CASE 6D

Appendix B

BUILDING DESCRIPTIONS

This appendix contains a description of each building, a photograph and plan of the floor-over-basement areas, a discussion and diagram of the floor systems analyzed, and a discussion of the upgrading potential, for ten of the 11 buildings examined in this study. One of the buildings was previously reported in Appendix A (as noted below). The building data are arranged by building number as follows:

A summary of the building element data for all floors analyzed is given in Table 1 in the main body of the report.

B.1 BLDG 110, HENRY R. LANDIS STATE HOSPITAL, PHILADELPHIA, PA

B.1.1 Description

The Henry R. Landis State Hospital, constructed in 1960 except for the westerly wing (designed in 1963), is located at 2100 South College Avenue, Philadelphia, Pennsylvania. The use class is 41, Government and Public Service, Hospital, and the building consists of seven stories (first of which is "Ground Floor" at zero to 100% below grade), plus a small partial basement (fully below grade). The overall height of the building is about 25 m (82 ft) (excluding a small penthouse and the partial basement), and plan dimensions of 64.6 m by 102.7 m (212 ft by 337 ft) provide an area of about 301.9 m^2 (3,250 ft^2) on the partial basement level, 3779 m^2 (40,680 ft^2) on the "Ground Floor" level [elevation 30 m (98.5 ft)], and 3452 m^2 (37,160 ft^2) on the first floor [elevation 34.3 m (112.5 ft)] aboveground and most higher levels. Figure 16 shows the exterior walls and plan of the building at the first floor level.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of R/C one-way slabs, beams, columns, and footings. Since the properties of the reinforcing steel were not given in the survey data, it was assumed for the analysis that the steel has a mean dynamic yield strength of 365.4 MPa (53,000 psi). Specified concrete 28-day strength was 22.8 MPa (3,300 psi) [30.3 MPa (4400 psi) for columns in original building; 22.8 MPa (3300 psi) for new (westerly) hospital wing columns].

The exterior walls are constructed primarily of face brick, or concrete panels and large glass areas. The interior partitions are constructed mostly of metal.

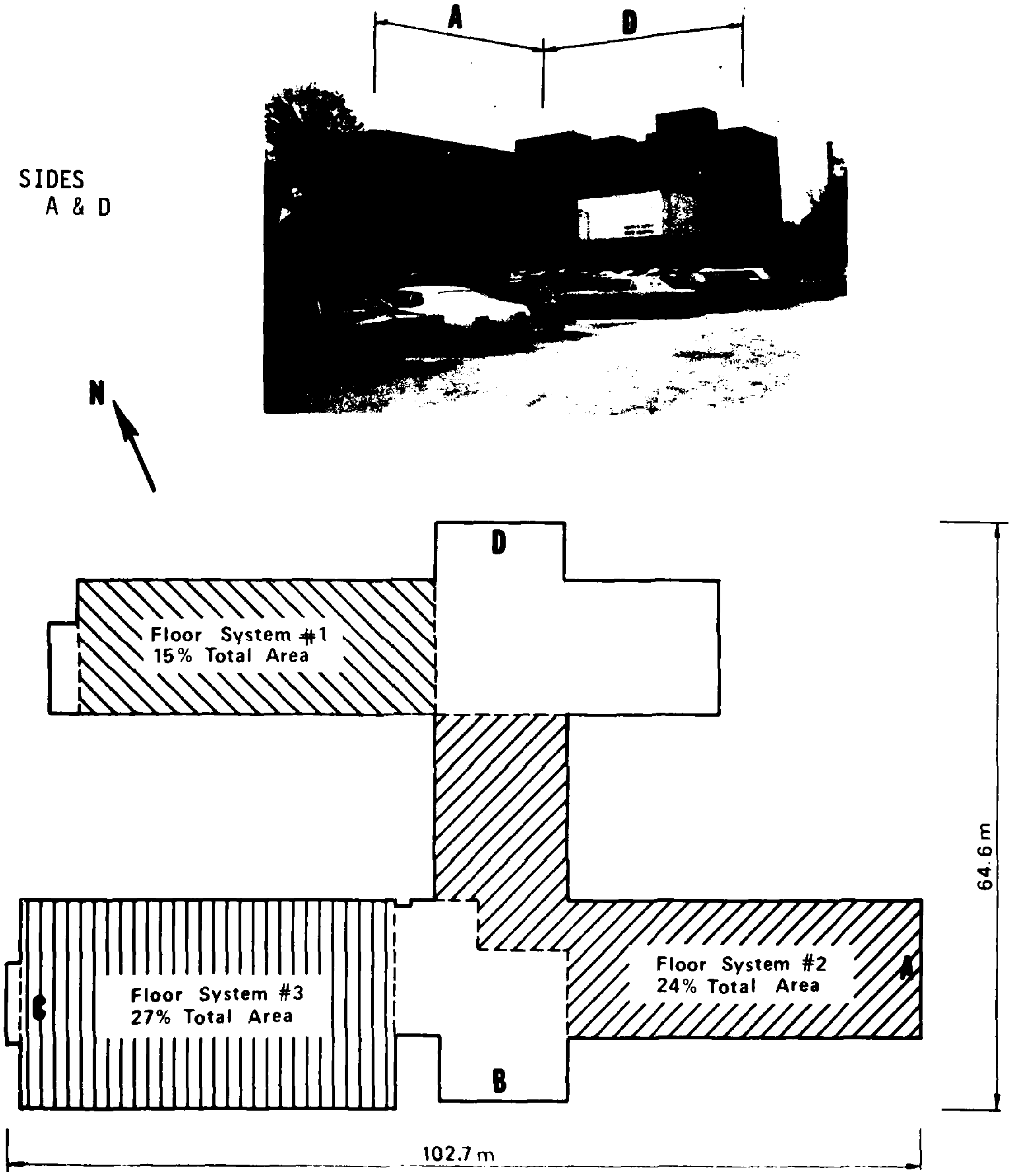


Figure 16: BLDG 110, HENRY R. LANDIS STATE HOSPITAL, Philadelphia, PA - Photograph and Plan of First Floor

B.1.2 Floor Systems

In Building 110, three floor systems were chosen for analysis. System #1 has a large clear span similar to Building 100. Systems #2 and #3 are typical of slab and beam construction. The location of the three systems and the percentage of the total usable basement shelter area represented are shown in Figure 16, while Figure 17 diagrams all three floor systems.

The results of the analysis of the MICO for the "as built" cases ranged between 22.3 and 53.7 kPa (3.2 and 7.8 psi). This is typical for the current sample of 11 NSS buildings. For system #1, the MICO was 28.6 kPa (4.2 psi); for system #2, 22.3 kPa (3.2 psi); and for system #3, 29.0 kPa (4.2 psi).

B.1.3 Upgrading Potential

For the slabs in floor systems #1 and #2, upgrading by the addition of one beam at mid-span yielded results of 93.5 and 46.7 kPa (13.6 and 6.8 psi) respectively. Steel detailing of the slab in system #3 precluded its upgrading; however, its "as-built" predicted MICO was above the upgraded value of the slab in floor system #2.

The beams of all three floor systems allowed for additional support columns at mid- or third-spans; however, the maximum MICOs for the three beams only ranged between 117.6 and 120.4 kPa (17.1 and 17.5 psi). One beam case, 6E, employed the upgrading scenario of adding a beam at mid-span to support both beam and slab. While providing a higher MICO than the regular added mid-span support case, 6B, this scenario is weaker than the added third-span support case (6C) also allowed by this beam [117.6 kPa (17.1 psi)].

The upgrading potential for this building was limited by the upgraded slab in system #2 to 46.7 kPa (6.8 psi).

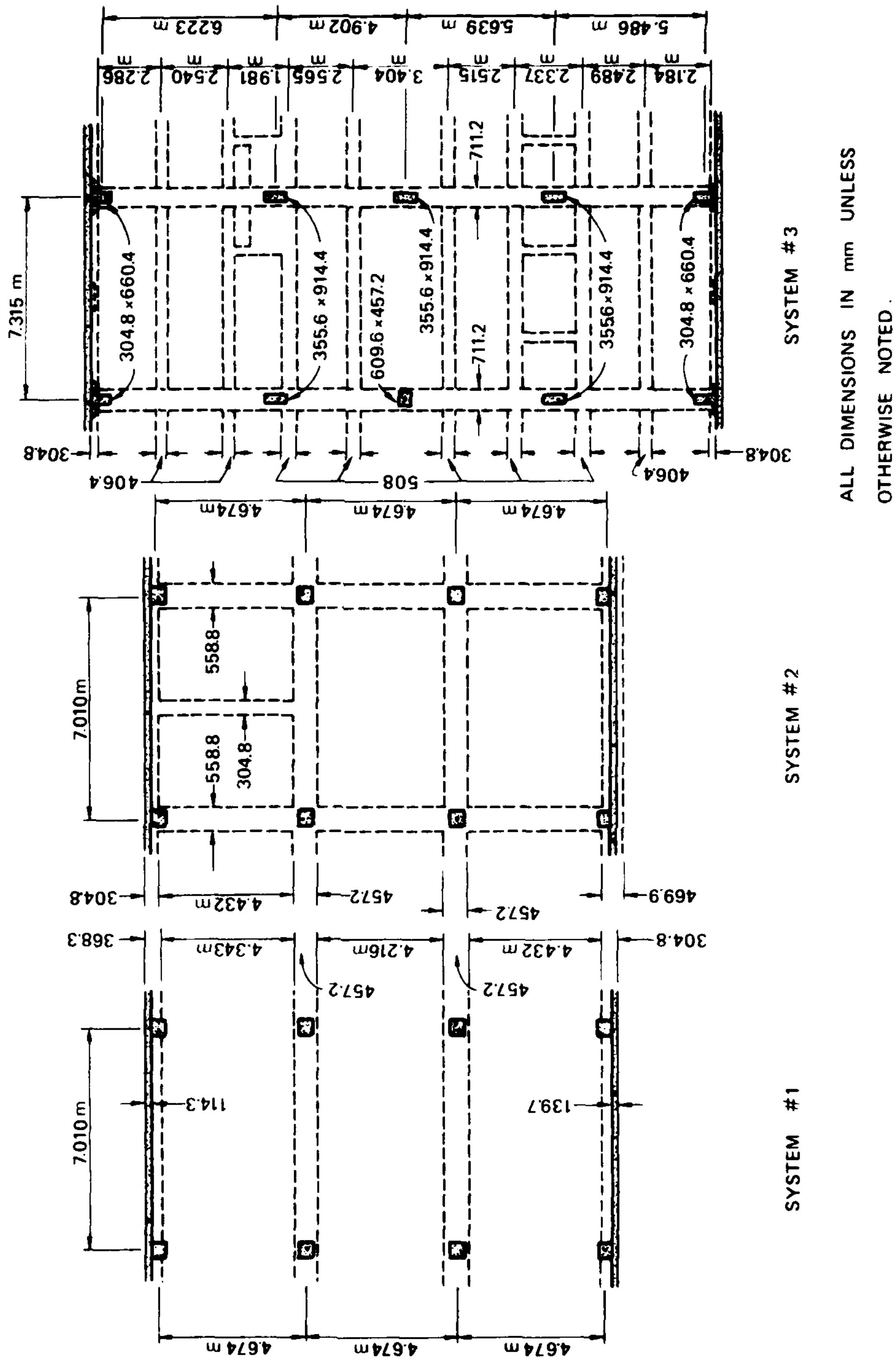


Figure 17: BLDG 110, HENRY R. LANDIS STATE HOSPITAL - Floor Systems

B.2 BLDG 111, GRANT BUILDING, PITTSBURGH, PA

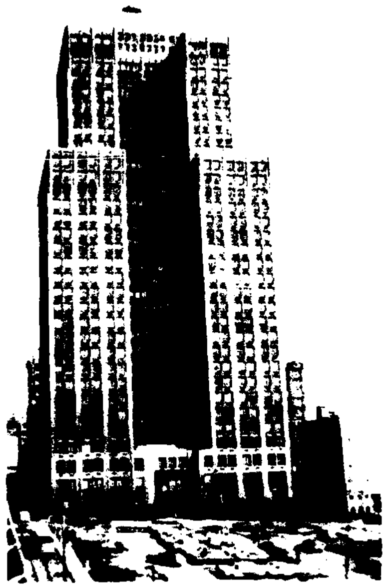
B.2.1 Description

The Grant Building, constructed in 1929, is located at 302 Grant Street, Pittsburgh, Pennsylvania. The use class is 51, Commercial, Offices (except Bank on one story), and the building consists of 37 stories and four basements. The overall height of the building is about 128 m (420 ft), and plan dimensions of 52.4 m by 38.7 m (172 ft by 127 ft) provide an area of about 1814 m^2 (19,530 ft^2) on the first floor level and about the same on the first basement level. Figure 18 shows the exterior walls and first floor plan of the building.

The building has a steel frame with rivetted (nonrigid-frame) column and beam connections. The floor system consists primarily of reinforced concrete (R/C) pan-joist construction. Since the properties of the structural steel were not given in the survey data, it was assumed for the analysis that the steel has a mean dynamic yield strength of 282.7 MPa (41,000 psi). R/C was assumed to have a concrete (28-day) mean dynamic strength of 17.2 MPa (2,500 psi) and a rebar mean dynamic strength of 303.4 MPa (44,000 psi).

The exterior walls are constructed of steel frame with brick facing (details unavailable). The non-bearing interior partitions are constructed of tile in stair and elevator cores (elsewhere not known).

SIDE A



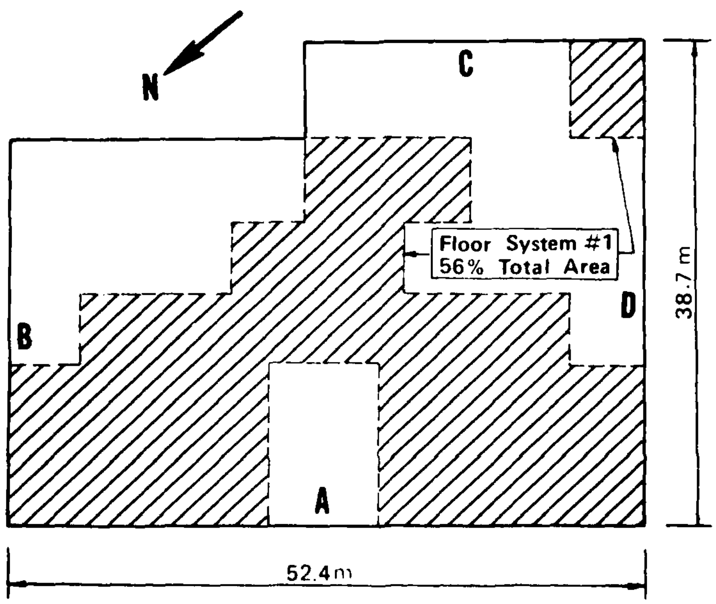


Figure 18: BLDG 111, GRANT BLDG, Pittsburgh, PA - Photograph and Plan of First Floor

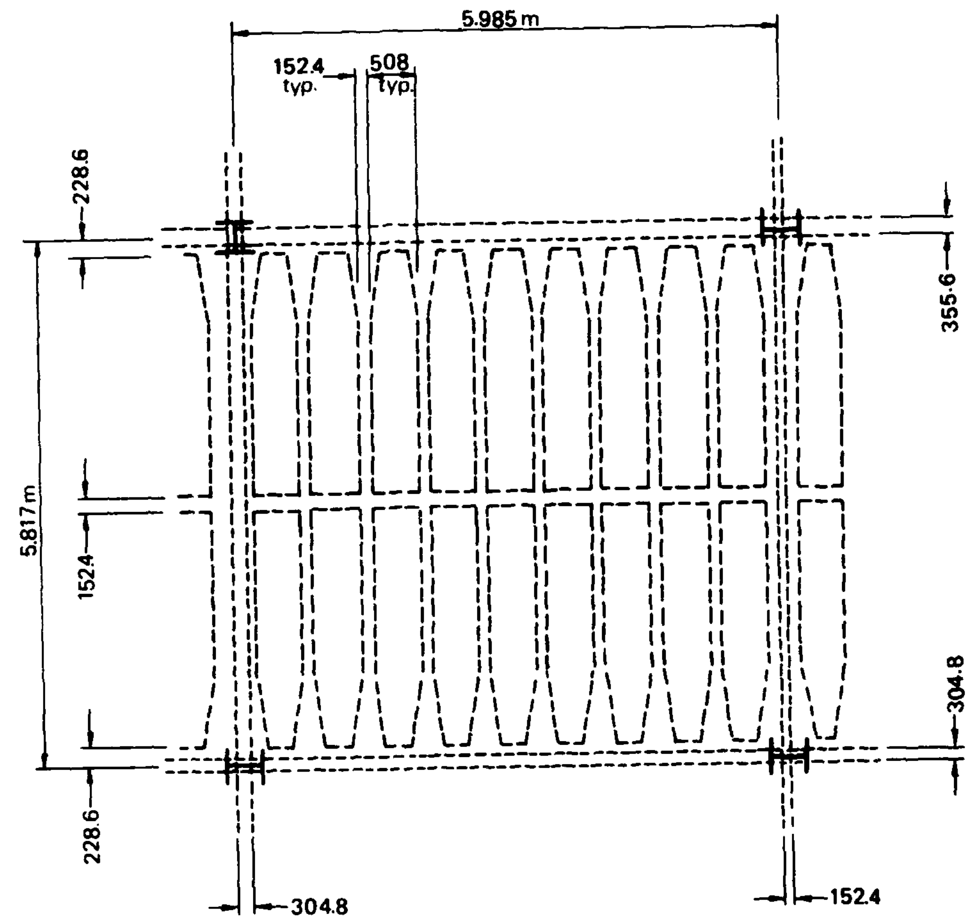
B.2.2 Floor Systems

Building 111 had one floor system chosen for analysis. It is a pan-joist system consisting of a slab, T-beam (joist), and steel girder. The location of the system and the percentage of the total usable basement shelter area it represents are shown in Figure 18. A diagram of the system is given in Figure 19.

The joist and girder were analyzed as having MICOs of 50.1 and 75.6 kPa (7.3 and 11 psi), respectively, for the "as built" case, while the slab showed 100.7 kPa (14.6 psi) using standard analysis and 466.0 kPa (67.6 psi) assuming fully restrained behavior. The floor system, therefore, has a MICO of 50.1 kPa (7.3 psi).

B.2.3 Upgrading Potential

Both the joist and girder allow for added supports at third-span intervals raising their predicted MICOs to 236.4 kPa (34.3 psi) and 626.3 kPa (90.8 psi), respectively. As with Building 100, no logical procedure was found to upgrade the slab portion of the system, but again the initial strength (based on the fully restrained model) was sufficient to match the upgraded strength of the other elements. In this building the basement upgrading potential was limited by the joist to 236.4 kPa (34.3 psi).



ALL DIMENSIONS IN mm UNLESS OTHERWISE NOTED

Figure 19: BLDG 111, GRANT BUILDING - Floor System

B.3 BLDG 136, FIRST FEDERAL SAVINGS & LOAN ASSN., AUGUSTA, GA

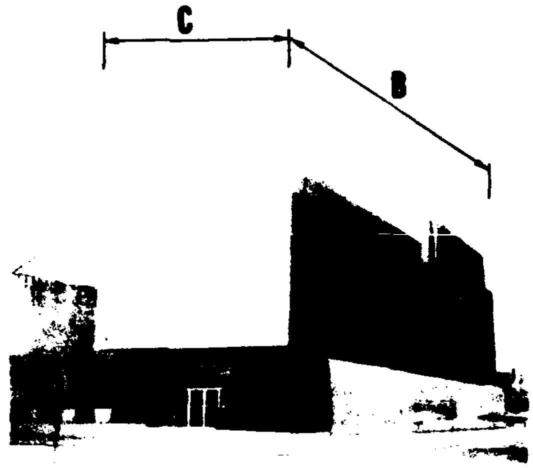
B.3.1 Description

The First Federal Savings and Loan Association building, constructed in 1960, is located at 985 Broad Street, Augusta, Georgia. The use class is 55, Commercial, Banks/Financial Institutions, and the building consists of 4 stories and a partial basement. The overall height of the building is about 20.4 m (67 ft), and gross plan dimensions of 18.3 m by 52.7 m (60 ft by 173 ft) provide a net area of about 768.2 m^2 (8,269 ft^2) on the partial basement level, about 855.2 m^2 (9,205 ft^2) on the first floor level; about 644.1 m^2 (6,933 ft^2) on the next two levels. Figure 20 shows the exterior walls at the first floor level.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of R/C pan-joist floors supported by R/C girders, outer walls, columns and footings. Structural steel used was A7, and rebars were intermediate grade for which a mean dynamic yield strength of 365.4 MPa (53,000 psi) was assumed. Concrete 28-day strength was specified as 20.7 MPa (3,000 psi).

The lower exterior walls were constructed primarily of brick, but with some large glazed areas. The interior partition construction varied widely.

SIDES
B & C



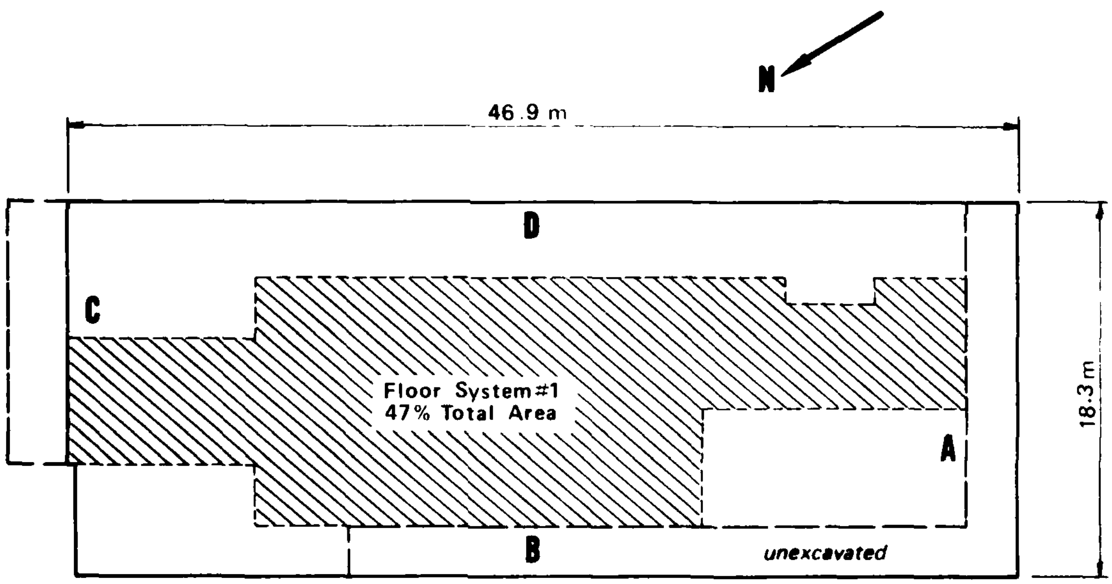


Figure 20: BLDG 136, FIRST FEDERAL SAVINGS & LOAN ASSN., Augusta, GA - Photograph and Plan of First Floor

B.3.2 Floor Systems

Building 136 had one floor system chosen for analysis. It is a pan-joist system consisting of a slab, T-beam (joist), and reinforced concrete girder. The location of the system and the percentage of the total usable basement shelter area it represents are shown in Figure 20, while the system is diagramed in Figure 21.

The joist and girder were analyzed as having MICOs of 24.3 and 29.6 kPa (3.5 and 4.3 psi), respectively, for the "as built" case. The slab was analyzed at 85.5 kPa (12.4 psi) using standard analysis and at 488.6 kPa (70.9 psi) assuming fully restrained behavior for the "as built" case. The joist at 24.3 kPa (3.5 psi) determined the strength of the floor system.

B.3.3 Upgrading Potential

The joist only allowed for one additional support at the mid-span increasing its predicted MICO to 66.9 kPa (9.7 psi). The girder allowed added third-span supports for a MICO of 142.7 kPa (20.7 psi). A special upgrading scenario for the girder, case 3H, investigated adding a girder line to support the beams (but not the slabs) at mid-span. The result was weaker than either the regular added mid- or third-span support cases. Again, as in previous pan-joist systems, the "as built" case predictions had to suffice for the slabs. In Building 136, the upgrading potential was determined by the joist at 66.9 kPa (9.7 psi).

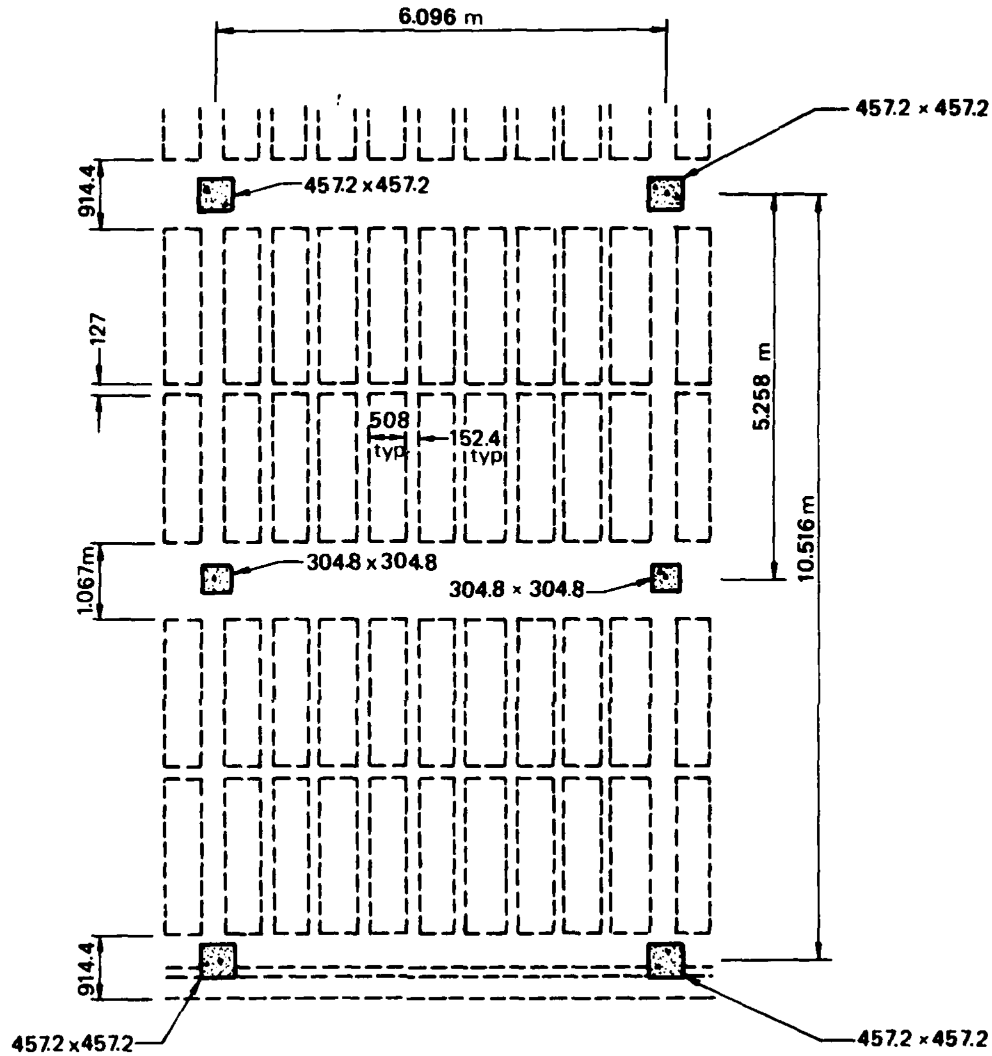


Figure 21: BLDG 136, FIRST FEDERAL SAVINGS & LOAN ASSN. - Floor System

B.4 BLDG 167, LAFAYETTE TOWERS BUILDING #2, DETROIT, MI

B.4.1 Description

The Lafayette Towers Building #2, constructed in 1962, is located at 1321 Orleans Street, Detroit, Michigan. The use class is 11, Residential, Apartment/Hotel, and the building consists of 21 stories, plus a basement, a mezzanine, and a penthouse. The overall height of the building is about 64.6 m (212 ft), and plan dimensions of 20.7 m by 64.0 m (68 ft by 210 ft) provide an area of about 1276 m^2 (13,690 ft^2) on the basement level and 1185 m^2 (12,750 ft^2) on the levels above the ground floor. Figure 22 shows the exterior walls and plan of the building.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of R/C flat plates. The properties of the specified reinforcing steel were given on the drawings, as were those of the concrete; mean dynamic yield strengths used were 365.4 and 25.9 MPa (53,000 and 3,750 psi), respectively.

The exterior walls are constructed primarily of glass, with narrow metal mullions and small horizontal metal plates, on a steel frame structure. The interior partitions are constructed of unreinforced solid concrete block or laminated rocklath.

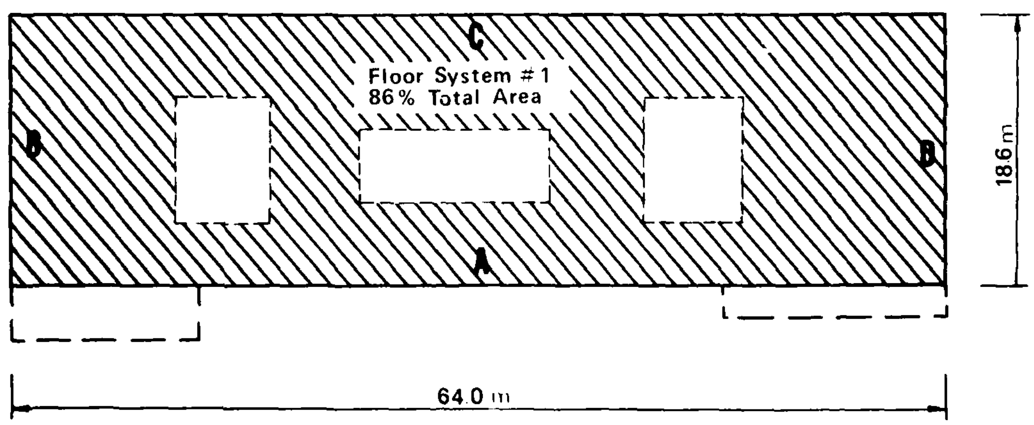


Figure 22: BLDG 167, LAFAYETTE TOWERS BLDG. #2, Detroit, MI - Photograph and Plan of First Floor

B.4.2 Floor Systems

The one floor system analyzed in Building 167 was a reinforced concrete flat plate system. The location of the system and the percentage of the total usable basement shelter area it represents are shown in Figure 22. Figure 23 provides a diagram of this system. The initial analysis of the slab measured the MICO under both regular conditions--43.5 kPa (6.3 psi)--and shear failure--11.2 kPa (1.6 psi). (The prediction for shear failure seems extremely low, and we are reviewing that portion of the computer program for errors.)

B.4.3 Upgrading Potential

Upgrading scenarios for a flat plate system include added columns at mid-points which give predictions of 212.6 kPa (30.8 psi) under regular conditions and 60.5 kPa (8.8 psi) under shear failure (again suspiciously low); added support beams between columns, which yield a prediction of 63 kPa (9.1 psi); and, added support beams between columns and at mid-points between columns, which yield a prediction of 137.2 kPa (19.9 psi).

The upgrading potential of this building could be as low as 60.5 kPa (8.8 psi) if shear is considered or as high as 212.6 kPa (30.8 psi) if it is not. In the statistical comparison section, 212.6 kPa (30.8 psi) was the value assigned this building.

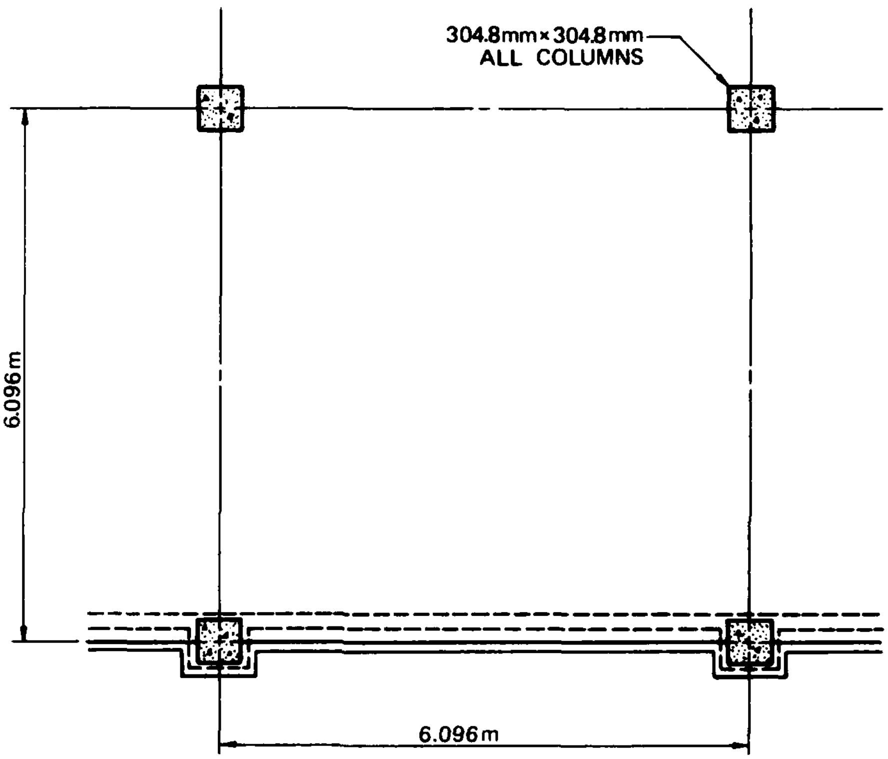


Figure 23: BLDG 167, LAFAYETTE TOWERS BUILDING #2 - Floor System

B.5 BLDG 188, STATE WILDLIFE CONSERVATION BLDG, OKLAHOMA CITY, OK

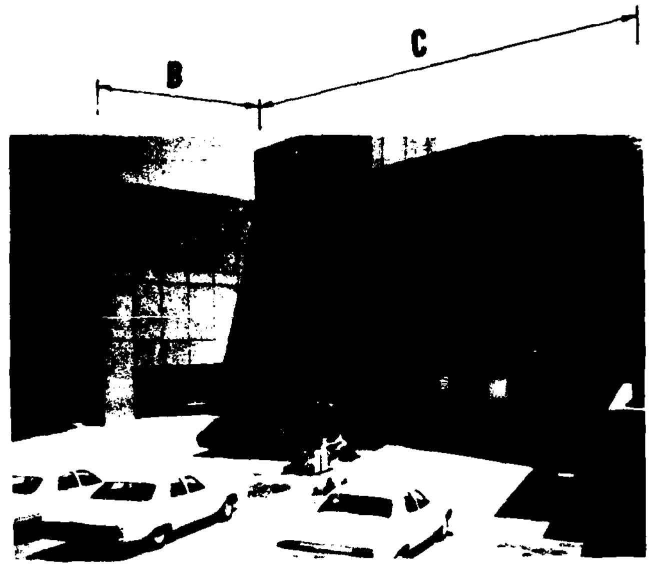
B.5.1 Description

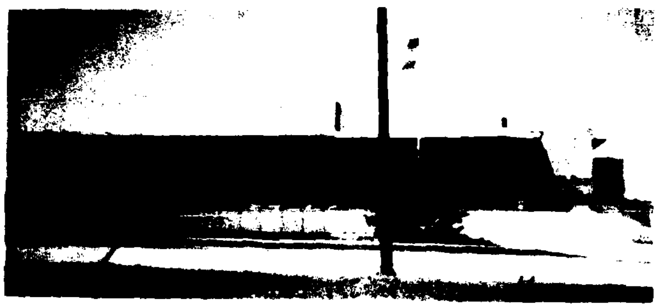
The State Wildlife Conservation Building, built in 1965, is located at 1801 Lincoln Boulevard, Oklahoma City, Oklahoma. The use class is 45, Government and Public Service, Offices, and the building consists of two stories and a basement. The overall height of the building is about 11.3 m (37 ft), and plan dimensions of 20.1 m by 62.8 m (66 ft by 206 ft) provide an area of about 721.9 m^2 (7,771 ft^2) on the partial basement level, 803.2 m^2 (8,646 ft^2) on the first floor level, and 774.6 m^2 (8,338 ft^2) on the second floor level. Figure 24 shows the exterior wall line at the first floor level.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of 152.4 mm (6 in.) thick R/C one-way slabs supported by R/C band beams, columns, and footings, in bents with two unequal spans. The few structural steel shapes used are A-36; reinforcing steel was specified as A-15 and A-305 for which the analyses used a mean dynamic yield strength of 303.4 MPa (44,000 psi). Concrete was specified to be 20.7 MPa (3,000 psi) (28-day strength) for which a mean dynamic yield strength of 25.9 MPa (3,750 psi) was used.

The exterior walls are constructed primarily of precast concrete (p/c) panels, except that the front wall (side A) has p/c fins supporting large glass panes. The interior partitions are constructed of either clay tile or metal studs and plaster. Basement R/C exterior walls with pilasters, and R/C columns and footings, were all judged from experience with other analyses to be adequate to exploit the inherent blast resistance of the first floor slab described above.

SIDES
B & C





SIDE A

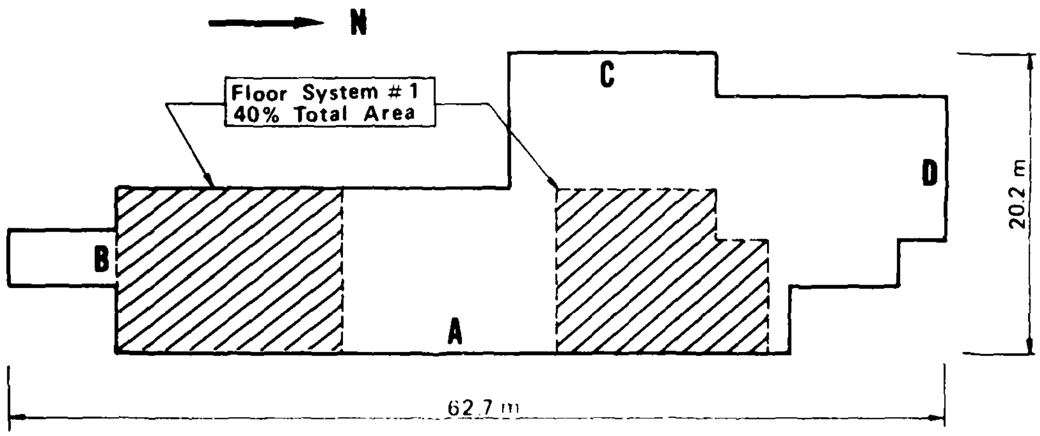


Figure 24: BLDG 188, STATE WILDLIFE CONSERVATION BLDG, Oklahoma City, OK - Photograph and Plan of First Floor

B.5.2 Floor Systems

Building 188 had one floor system chosen for analysis; it consists of a slab and two beams. The location of the system and the percentage of the total usable basement shelter area it represents are given in Figure 24, while a diagram of the system is given in Figure 25.

The "as built" case analysis predicts MICOs of 72.5 kPa (10.5 psi) for the slab, 51.4 kPa (7.5 psi) for the shorter beam and 30.8 kPa (4.5 psi) for the longer beam. The floor system strength is determined by the longer beam at 30.8 kPa (4.5 psi).

B.5.3 Upgrading Potential

The slab and shorter beam allowed upgrading only at the mid-spans producing predictions of 143.8 kPa (20.9 psi) and 161.5 kPa (23.4 psi), respectively. The long beam was able to reach 168.9 kPa (24.5 psi) with added third-span supports. Building basement potential was 143.8 kPa (20.9 psi) based on the upgraded slab.

Other scenarios tried included adding a support beam to halve the loaded area on the short beam, 2H, and adding a support beam and a support column at mid-span for the short beam, 2I. These resulted in MICOs of 78.2 and 102.9 kPa (11.3 and 14.9 psi), respectively.

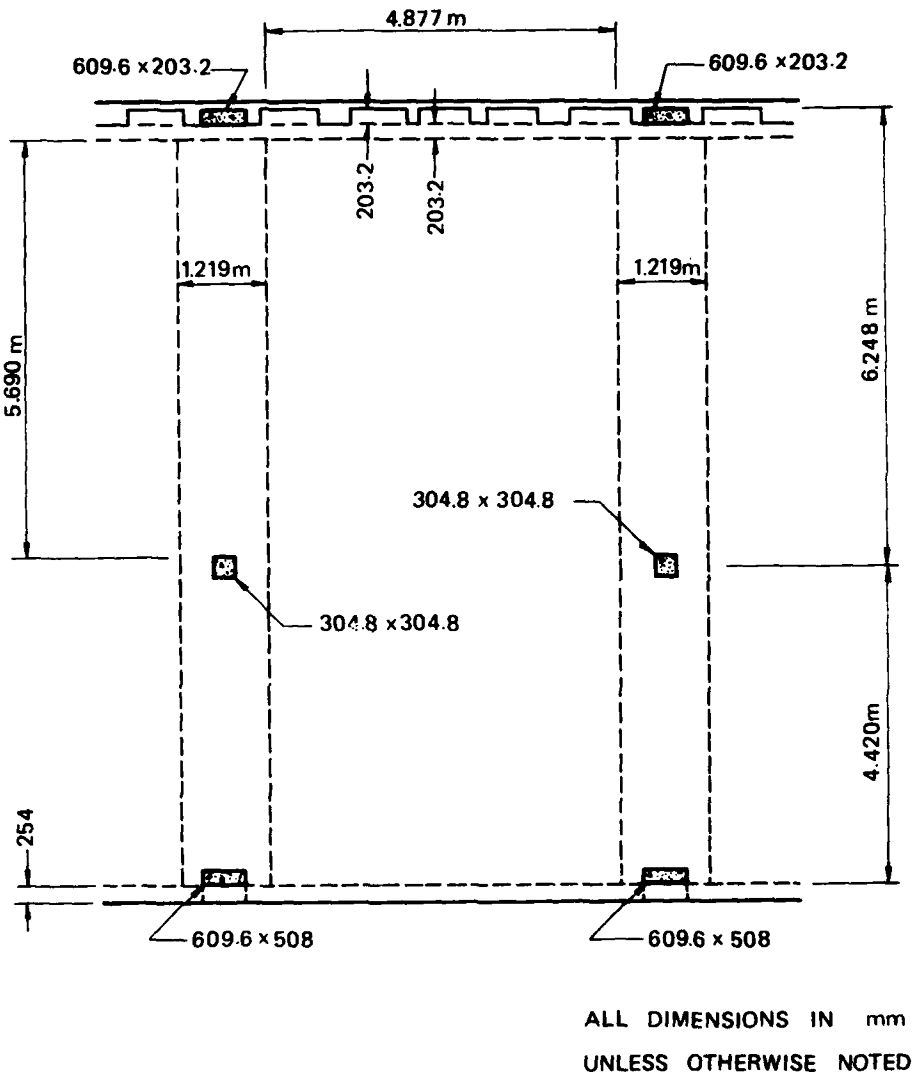


Figure 25: BLDG 188, STATE WILDLIFE CONSERVATION BLDG - Floor System

B.6 BLDG 200, FITZSIMONS GENERAL HOSPITAL, DENVER, CO

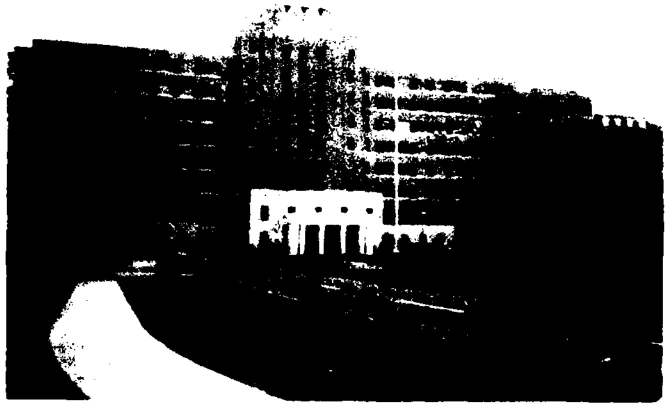
B.6.1 Description

Fitzsimons General Hospital, constructed in 1940, is located on Bruns Avenue in Denver, Colorado. The use class is 41, Government and Public Service, Hospital, and the building consists of ten stories and a partial basement. The overall height of the building is about 43.3 m (142 ft), and plan dimensions of 167.9 m by 89.3 m (551 ft by 293 ft) provide an area of about 1709 m^2 (18,397 ft^2) on the partial basement level and 5892 m^2 (63,426 ft^2) on the next two aboveground levels. Figure 26 shows the exterior walls and plan of the building at the first floor level.

The building has a reinforced concrete (R/C) frame with conventional nonrigid-frame column and beam connections. The floor system consists primarily of R/C one-way slabs, beams, girders, columns, and footings. Since the properties of the reinforcing steel were not given in the survey data, it was assumed for the analysis that the steel has a mean dynamic yield strength of 303.4 MPa (44,000 psi).

The exterior walls are constructed primarily of unreinforced brick and masonry, anchored into R/C spandrel beams. The interior partitions are plastered, except that rest rooms, labs, operating rooms, etc., are covered with ceramic or glazed tile. Basement exterior walls, columns, pilasters, floor slabs, and footings are R/C, as are those basement interior walls that separate the standing-height basement areas from the remaining areas (pipe spaces).

SIDE A



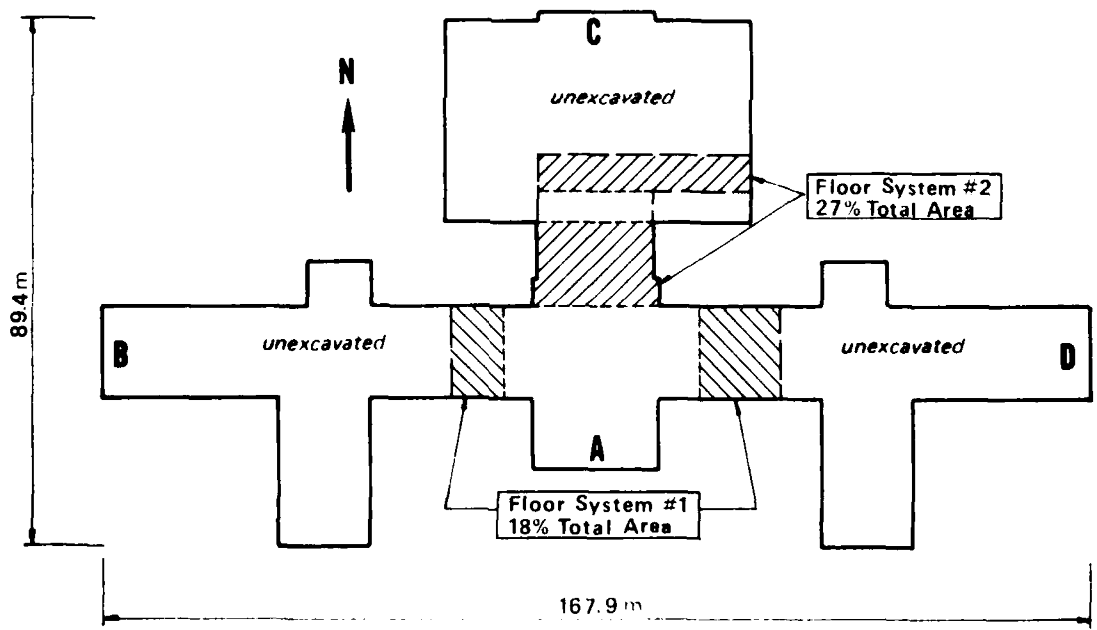


Figure 26: BLDG 200, FITZSIMONS GENERAL HOSPITAL, Denver, CO - Photograph and Plan of First Floor

B.6.2 Floor Systems

Two floor systems were found for analysis in Building 200. The first comprised a slab, beam and girder; the second a slab and two beams. The locations of the systems and the percentage of the total usable basement shelter area they represent are given in Figure 26, while Figure 27 diagrams the two systems.

The initial analyses for all elements ranged between 22.6 kPa and 45.0 kPa (3.3 and 6.5 psi), except for the second beam in system #2. This short beam, too small to upgrade, had a prediction of 83.9 kPa (12.2 psi). Floor system #1 had a MICO of 33 kPa (4.8 psi) and floor system #2 had a MICO of 22.6 kPa (3.3 psi).

B.6.3 Upgrading Potential

The slabs (one-way) in both systems have short spans too short to permit upgrading by added beams parallel to the long span; however, the slabs did allow one added beam parallel to the short span (cases 1e and 4e). This upgrading scenario, however, produces only small increases in strength from 45.0 to 60.5 kPa (6.5 to 8.8 psi) for the slab in system #1 and from 22.6 to 41.8 kPa (3.3 to 6.0 psi) for the slab in system #2.

The beam in system #1 permitted support columns at mid-span and third-span yielding predictions of 95.6 kPa (13.9 psi) and 119.9 kPa (17.4 psi). The girder in system #1 had as one upgrading scheme, 3H, the addition of a girder to support the beams in the system. The resulting MICO prediction of 59 kPa (8.6 psi) was far less than that produced by either mid-span or third-span added support columns--117.8 and 196.5 kPa (17.1 and 28.5 psi), respectively.

The first beam in system #2 reached a high of 128.6 kPa (18.7 psi) based on the addition of a beam at mid-span supporting both the slab and the beam, 5E. The second beam, as mentioned previously, was too short to upgrade.

For the entire building, the limiting case was the slab in system #2 which only reached 41.8 kPa (6.0 psi) after upgrading.

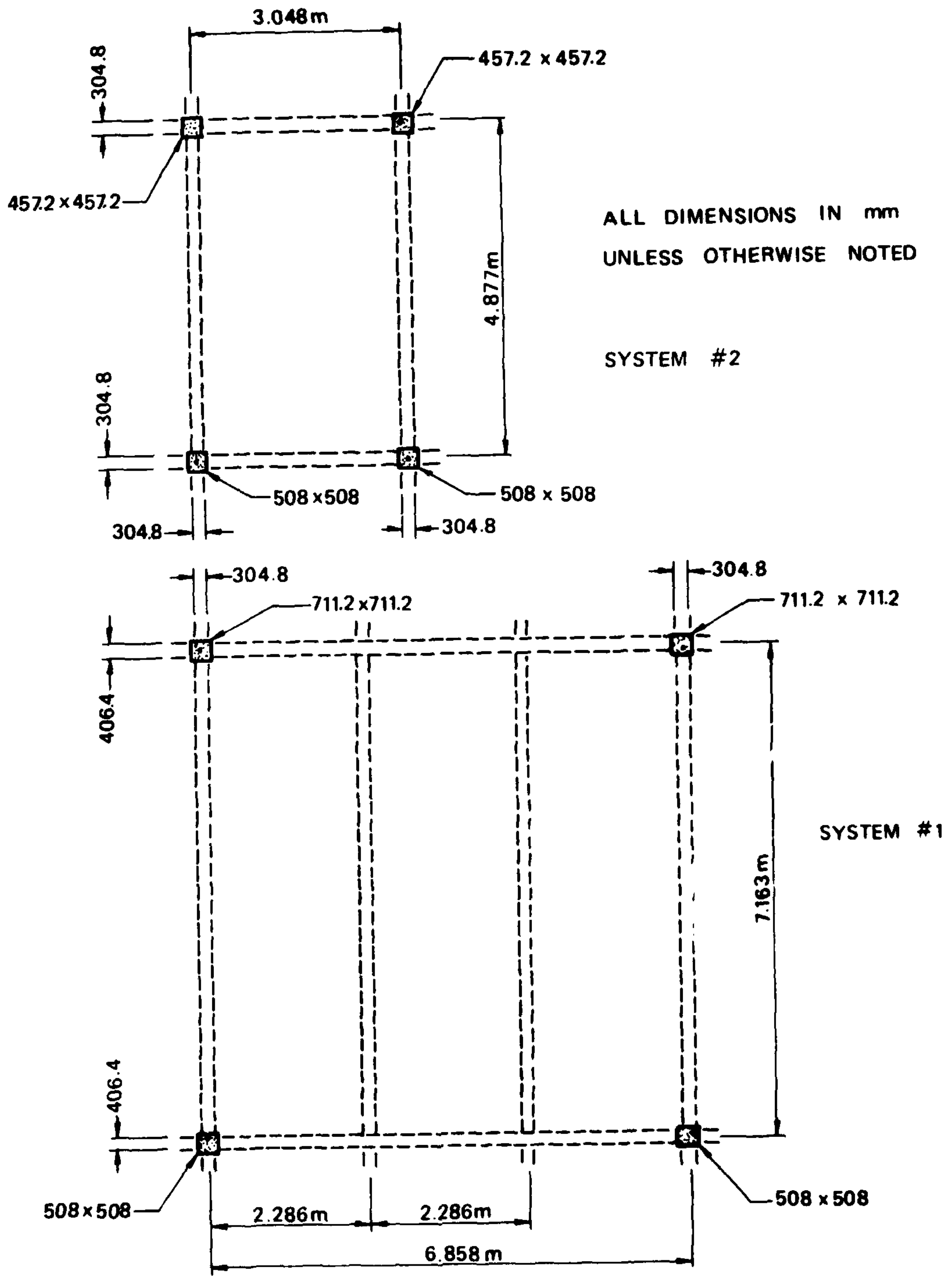


Figure 27: BLDG 200, FITZSIMONS GENERAL HOSPITAL - Floor Systems

B.7 BLDG 220, FIDELITY FEDERAL PLAZA, LONG BEACH, CA

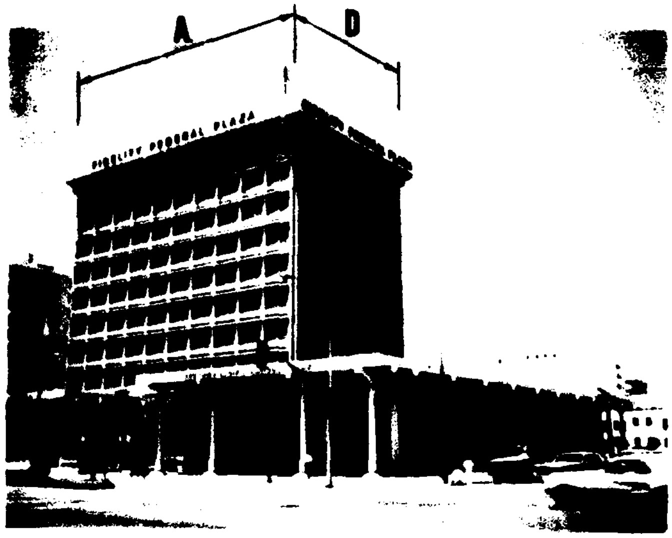
B.7.1 Description

The Fidelity Federal Plaza building, constructed in 1967, is located at 525 East Ocean Boulevard, Long Beach, California. The use classes are 55/53/51, Commercial: Banks/Financial Institutions (1st floor); Stores (non-food) (1st floor); Offices (higher floors); Parking (basements). The building consists of 11 stories, a basement and a sub-basement. The overall height of the building aboveground is about 43.6 m (143 ft), and plan dimensions of 74.4 m by 95.7 m (244 ft by 314 ft) provide an area of about 6420 m^2 (69,100 ft^2) on the basement level and on the aboveground levels above the first. The first level uses overhangs and glass walls, and is atypical of all other floors. Figure 28 shows the exterior walls and plan of the building.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of R/C one-way slabs, beams and girders. The reinforcing steel was specified as intermediate grade A15 and A305, for which the analyses used a mean dynamic yield strength of 365.4 MPa (53,000 psi) (CA432 for #14 and #18 rebars). Concrete for slabs, beams, girders, pilings, footings, and walls was specified as 20.7 MPa (3,000 psi) (28-day compressive strength); and that for columns (below "Ground Floor") as 27.6 MPa (4,000 psi).

The exterior walls are constructed primarily of metal furring channels, with masonry or concrete covering. The interior partitions are constructed primarily of metal studs, with drywall or plaster covering.

SIDES
A & D



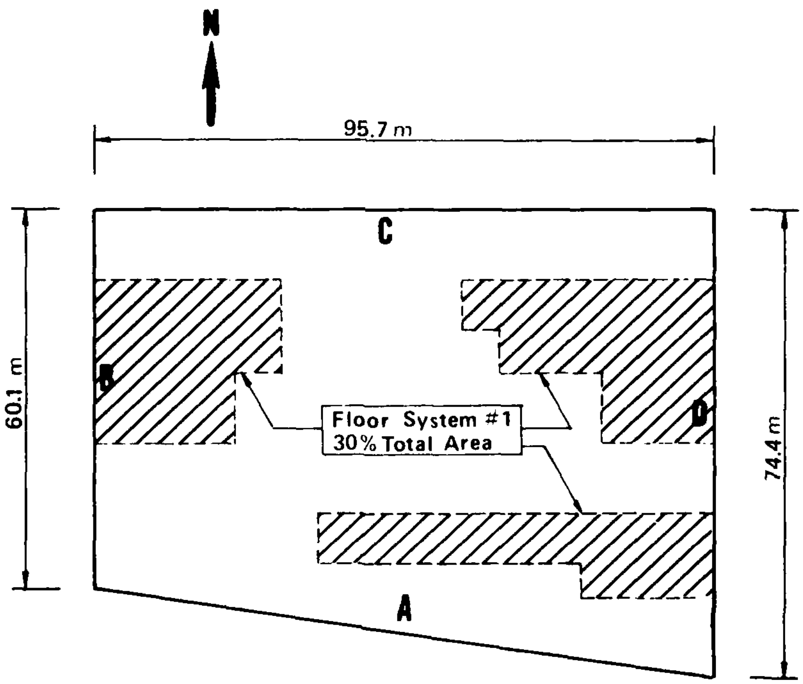


Figure 28: BLDG 220, FIDELITY FEDERAL PLAZA, Long Beach, CA - Photograph and Plan of First Basement (Upper Parking Level)

B.7.2 Floor Systems

One floor system was chosen for analysis; it consists of a slab, beam and R/C girder combination. The location of the system and the percentage of the total usable basement shelter area it represents are given in Figure 28. A diagram of the system appears in Figure 29.

Analysis of the "as built" case for the slab, beam, and girder produced MICOs of 57.2, 37.9, and 25.7 kPa (8.3, 5.5, and 3.7 psi), respectively. The floor system MICO was 25.7 kPa (3.7 psi).

B.7.3 Upgrading Potential

As in Building 200, the one-way slab has a short span too short for upgrading parallel to the long span, but did permit added beams at mid- or third-spans parallel to the short span (cases 1e and 1f). These produced only small increases in the MICO of the slab--62.0 kPa (9.0 psi) for added mid-span support and 67.6 kPa (9.8 psi) for added third-span support. Both the beam and girder allowed up to quarter-span intervals for the addition of support columns, raising their MICOs to 251.2 and 240.2 kPa (36.4 and 34.8 psi), respectively. The added girder scenario, 3H, only resulted in 54.5 kPa (7.9 psi).

Building basement potential was 67.6 kPa (9.8 psi) based on the highest upgrading possibility of the slab.

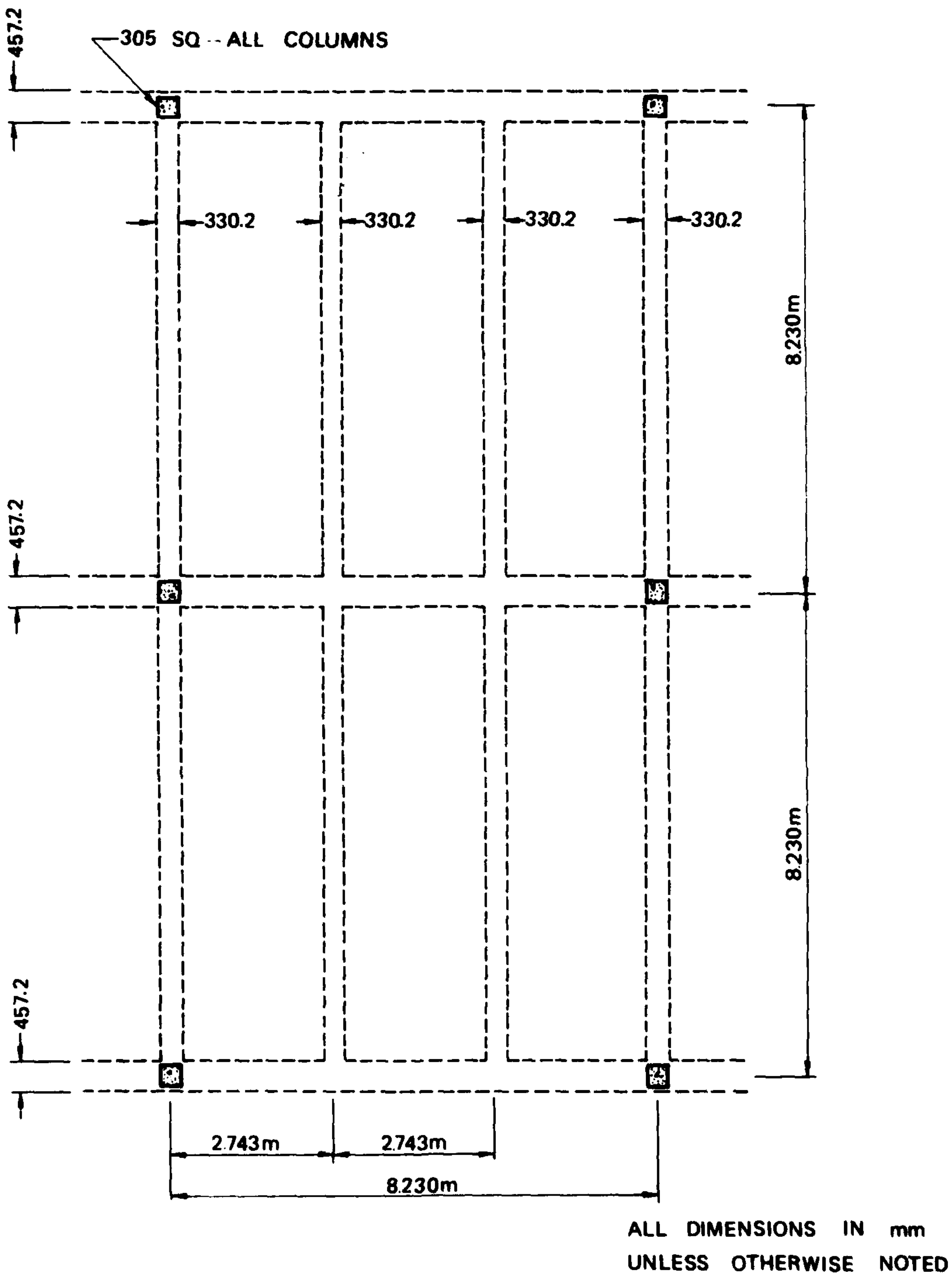


Figure 29: BLDG 220, FIDELITY FEDERAL PLAZA - Floor System

B.8 BLDG 225, BROADWAY CRENSHAW, LOS ANGELES, CA

B.8.1 Description

The Broadway Crenshaw building, constructed in 1947, is located at 4101 South Crenshaw Boulevard, Los Angeles, California. The use class is 53, Commercial, Stores Other Than Food Stores, and the building consists of three stories and a basement, plus a penthouse. The overall height of the building is about 22.3 m (73 ft), and first floor gross plan dimensions of 47.5 m by 109.1 m (156 ft by 358 ft) provide an area of about 4810 m^2 (51,770 ft^2) on the basement level and 4492 m^2 (48,350 ft^2) on the aboveground levels. Figure 30 shows the exterior walls and plan of the building.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of R/C flat slab construction, but with a substantial amount of one-way slab, beam and girder construction. Rebars were stated as intermediate grade; their strength was assumed for the analyses to have a mean dynamic yield value of 365.4 MPa (53,000 psi). Concrete strength was specified as 17.2 MPa (2,500 psi), 28-day test cylinder compression strength.

The exterior walls are constructed primarily of cast-in-place concrete (not load-bearing), most with stone veneer, with much glass on the first floor. The interior partitions are constructed variously of: concrete tile, cast-in-place concrete, and steel studs plastered both sides.

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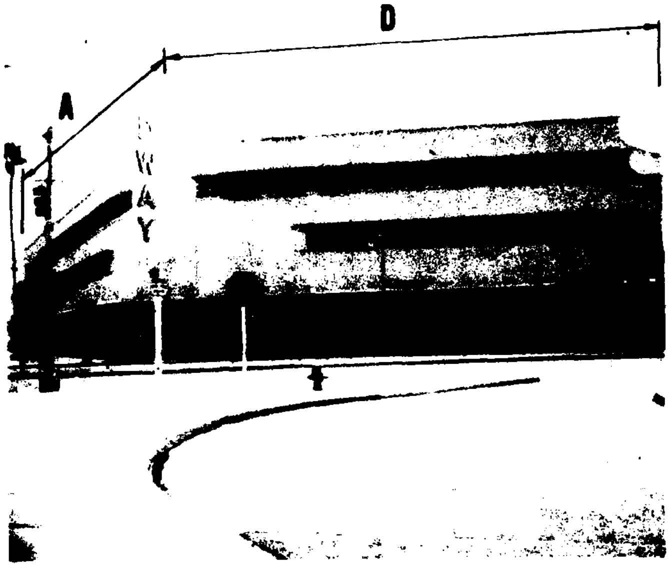
A & D



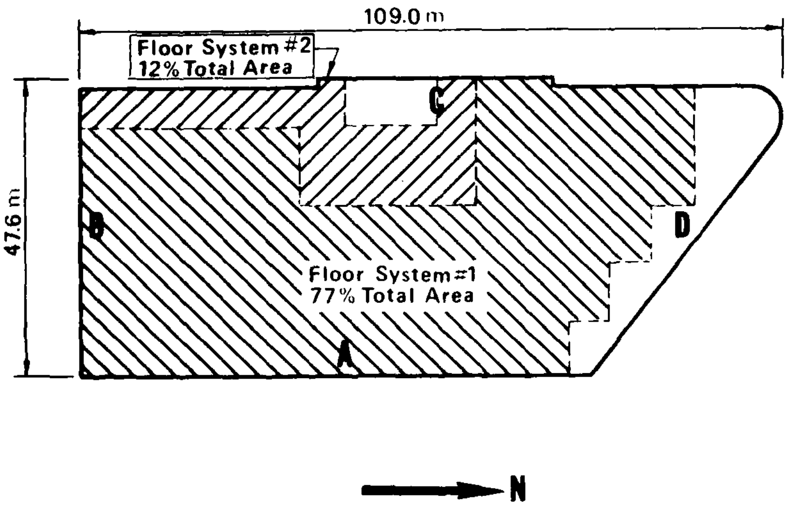


Figure 30: BLDG 225, BROADWAY CRENSHAW, Los Angeles, CA - Photograph and Plan of First Floor

B.8.2 Floor Systems

The two selected floor systems consist of a flat slab system and a slab, beam and girder system. The location of these systems and the percentage of the total usable basement shelter area they represent are shown in Figure 30. A diagram of system #2 appears in Figure 31.

In system #1, the results of the "as built" analyses showed the flat slab to have a MICO of 49.4 kPa (7.2 psi) under standard conditions and 47.4 kPa (6.9 psi) under shear failure. In system #2, the slab had a MICO of 77.8 kPa (11.3 psi), the beam 51.4 kPa (7.5 psi) and the girder 85.9 kPa (12.5 psi). Thus, system #1 had a MICO of 47.4 kPa (6.9 psi) based on the shear failure case, and system #2 had a MICO of 51.4 kPa (7.5 psi).

B.8.3 Upgrading Potential

The upgrading scenarios for system #1 added beams between columns and at mid-points between columns increasing the predicted MICO to 60.6 kPa (8.8 psi) for case 1i and 125.6 kPa (18.2 psi) for case 1j.

In system #2, the slab could only be upgraded by placing support beams parallel to the short span at mid-span or third-span. The resultant increase in MICOs was negligible: 86.9 kPa (12.6 psi) for case 2e and 89.2 kPa (12.9 psi) for case 2f. Both the beam and the girder had sufficient span to allow additional supports at third-span intervals. The resulting predictions were 323.6 and 473.1 kPa (46.9 and 68.6 psi), respectively. The poor upgrading potential of the slab in system #2 limited the overall building basement potential to 89.2 kPa (12.9 psi).

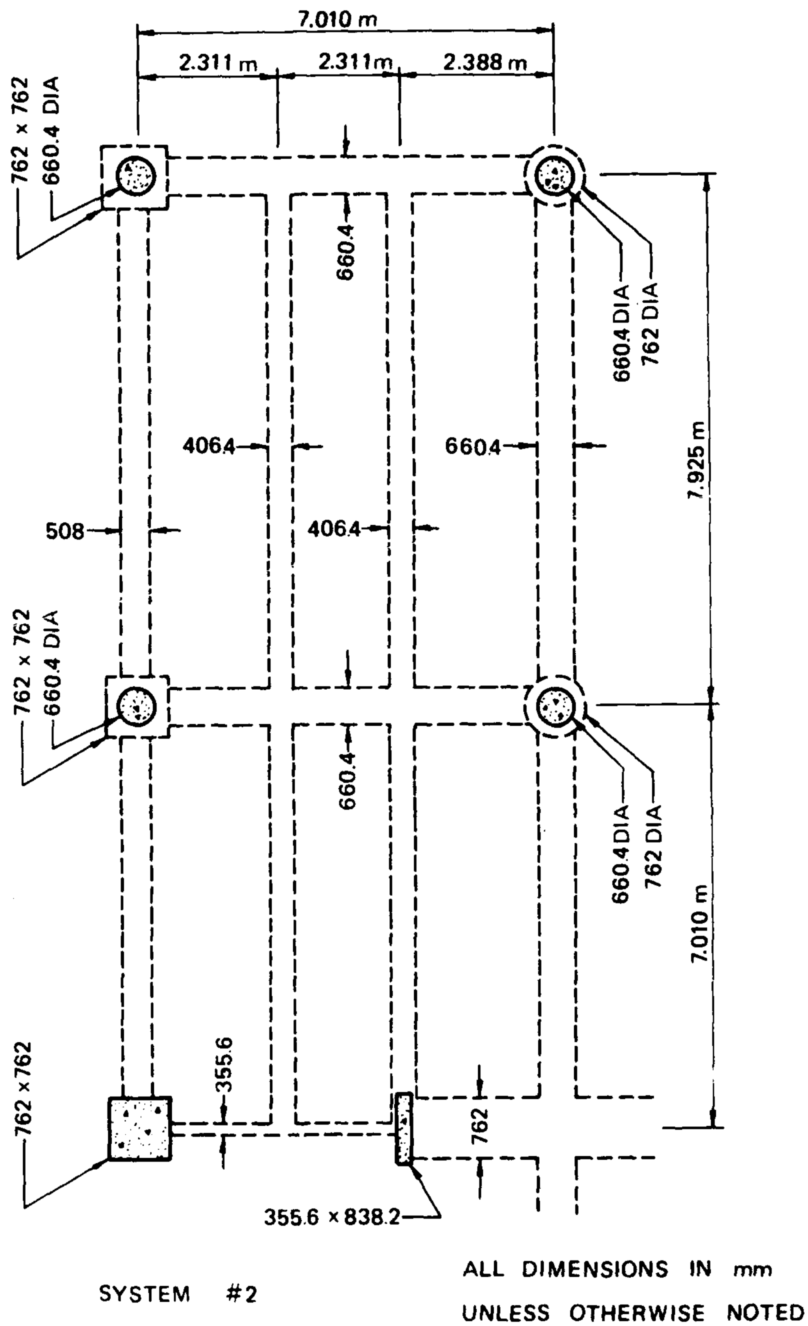


Figure 31: BLDG 225, BROADWAY CRENSHAW - Floor Systems

B.9 BLDG 227, MAY COMPANY EASTLAND SHOPPING CENTER, WEST COVINA, CA

B.9.1 Description

The May Company Eastland Shopping Center building, constructed in 1956, is located at 2831 East Garvey Avenue, West Covina, California. The use class is 53, Commercial, Stores Other Than Food Stores, and the building consists of 4 stories, 2 mezzanines, a partial basement and a penthouse. The overall height of the building is about 30.2 m (99 ft), and plan dimensions of 101.8 m by 116.4 m (334 ft by 382 ft) provide an area of about 1028 m^2 (11,060 ft^2) on the basement level and 9887 m^2 (106,420 ft^2) on the first aboveground level. Figure 32 shows a photograph and the first aboveground level, exterior wall plan of the building.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists of R/C pan-joist and flat slab construction over roughly equal areas, plus some 650 m^2 (7,000 ft^2) of one-way slab construction. Reinforcing steel was specified as A15, intermediate grade, with A82 for column spirals; it was assumed for the analysis that the A15 steel has a mean dynamic yield strength of 365.4 MPa (53,000 psi), with woven wire fabric at 386.1 MPa (56,000 psi). Concrete was specified as 20.7 MPa (3,000 psi) 28-day test strength.

The exterior walls are constructed primarily of concrete, but with some brick. The interior partitions are constructed of concrete block or timber-studwall.

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SUMMARY OF DYNAMIC ANALYSES OF SELECTED NSS BUILDINGS.(U)
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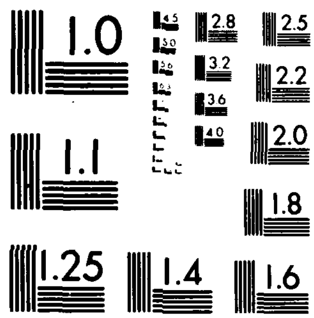

MICROCOPY RESOLUTION TEST CHART
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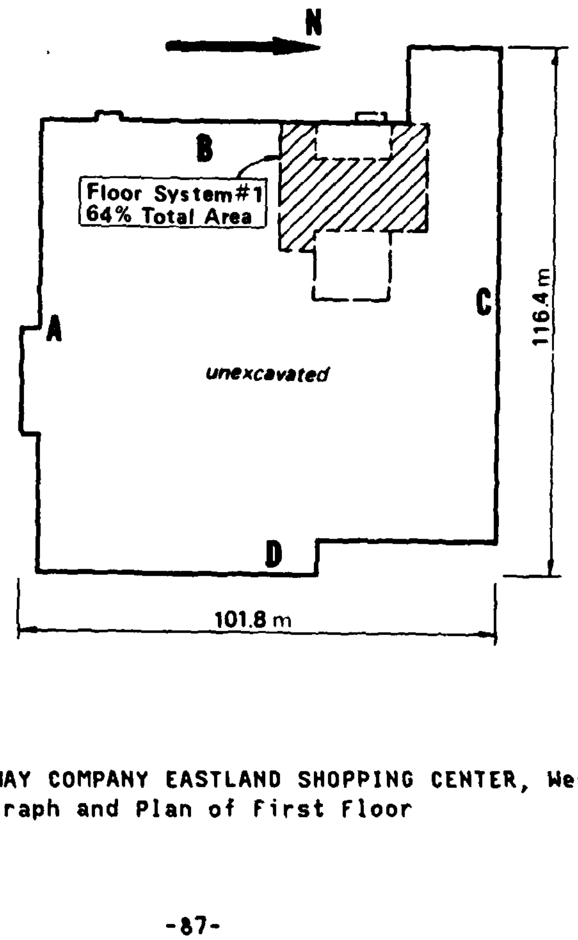

Figure 32: BLDG 227, MAY COMPANY EASTLAND SHOPPING CENTER, West Covina, CA - Photograph and Plan of First Floor

B.9.2 Floor Systems

One floor system was analyzed: a pan-joist system having a slab, T-beam (joist), and two girders. The location of the system and percentage of the total usable basement shelter area it represents are shown in Figure 32. A diagram of the system is shown in Figure 33.

For the "as built" case the slab portion of the system had MICOs of 70.3 kPa (10.2 psi) using the unrestrained slab model and 355.1 kPa (51.5 psi) using the fully restrained model. The joist prediction was 24.1 kPa (3.5 psi) and the two girders were calculated at 52.2 and 46.4 kPa (7.6 and 6.7 psi). For the floor system, the MICO was 24.1 kPa (3.5 psi).

B.9.3 Upgrading Potential

As discussed for other buildings, there are no upgrading scenarios for the slab in a pan-joist system. The fully restrained "as built" prediction of 355.1 kPa (51.1 psi) was used for the slab value. The joist and two girders permit the addition of support columns at third-span intervals with resulting MICOs of 151.1, 211.9 and 199.3 kPa (21.9, 30.7 and 28.9 psi), respectively. While additional girders are a feasible scenario, 3H, the increased MICO values are far lower than for third-span supports, only 97.7 kPa (14.2 psi). For this building basement, the upgrading potential is 151.1 kPa (21.9 psi) based on the joist as the weakest member.

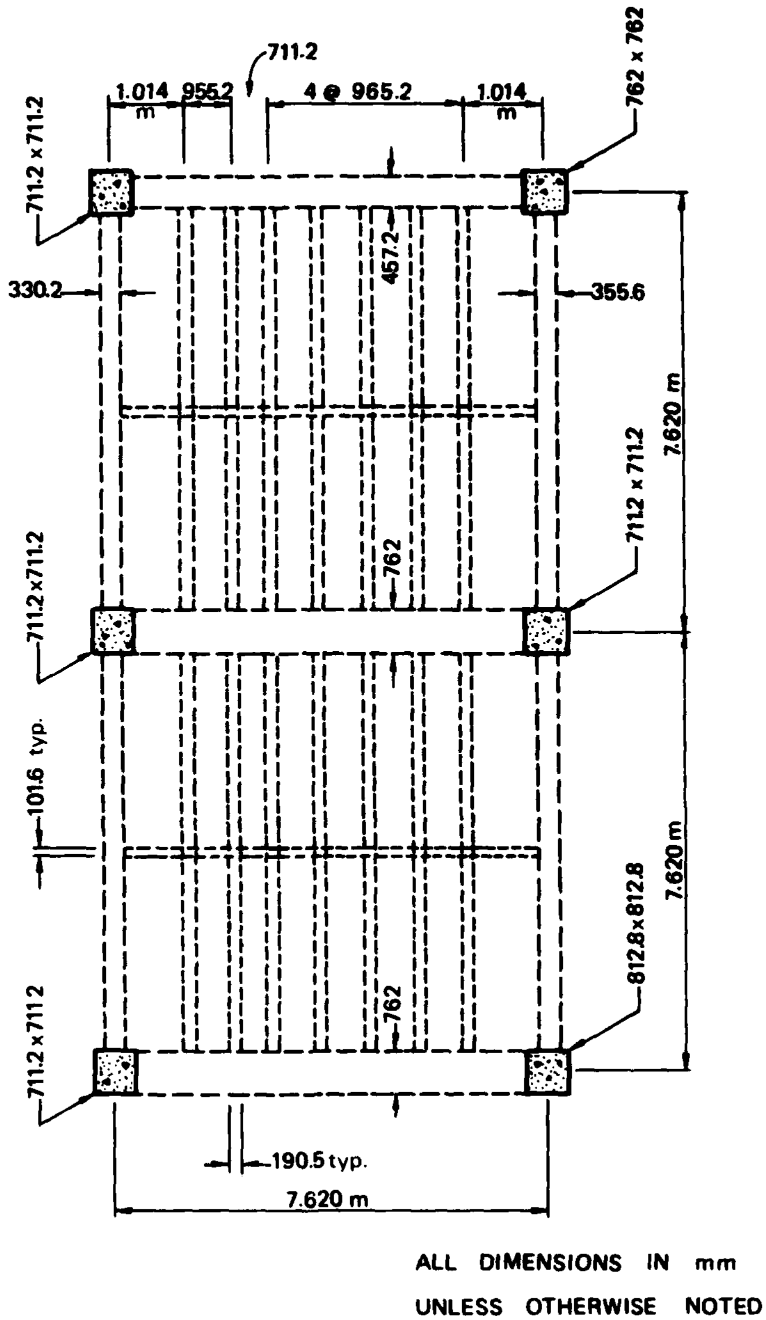

Figure 33: BLDG 227, MAY COMPANY EASTLAND SHOPPING CENTER - Floor System

B.10 BLDG 245, PORTLAND HILTON HOTEL, PORTLAND, OR

B.10.1 Description

The Portland Hilton Hotel, constructed in 1962, is located on 921 Southwest Sixth Avenue, Portland, Oregon. The use class is 11, Residential, Apartment/Hotel, and the building consists of 23 stories and 4 basements, plus a small penthouse. The overall height of the building is about 72.9 m (239 ft), and gross plan dimensions of 60.4 m by 61.0 m (198 ft by 200 ft) provide an area of about 3716 m^2 (40,000 ft^2) on the basement level, 3679 m^2 (39,600 ft^2) on the first level, and about 985 m^2 (10,600 ft^2) on other aboveground levels. Figure 34 shows photograph and first floor plan of the building.

The building has a reinforced concrete (R/C) frame with nonrigid-frame column and beam connections. The floor system consists primarily of pan-joist construction. The reinforcing steel is intermediate grade as shown on the building drawings, for which it was assumed that the steel has a mean dynamic yield strength of 365.4 MPa (53,000 psi); similarly, woven wire fabric was used at 386.1 MPa (56,000 psi) and concrete at 25.9 MPa (3,750 psi), dynamic strengths.

The exterior walls are constructed primarily of cast-in-place concrete (nonload-bearing) on the first floor, with much glass on upper floors. The interior partitions are constructed of cast-in-place concrete in the basement; unreinforced concrete block and cast-in-place concrete on the first floor; and wood studs and plasterboard on upper floors.

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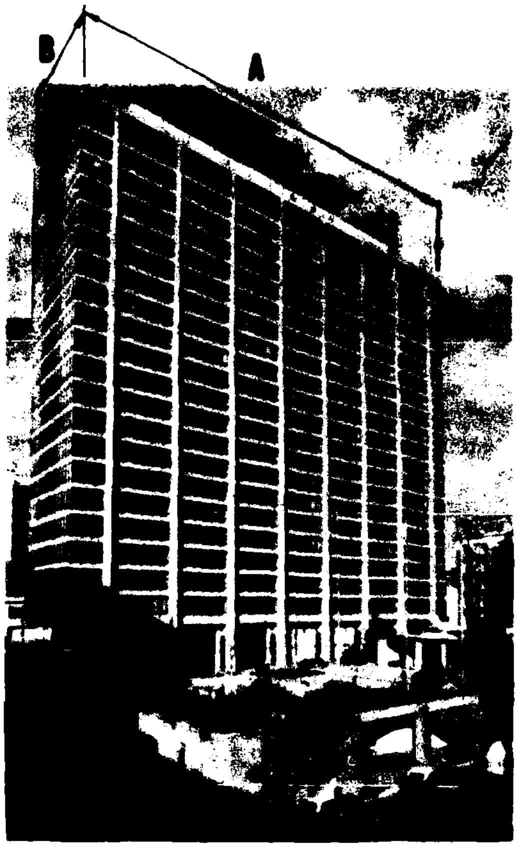
A & B



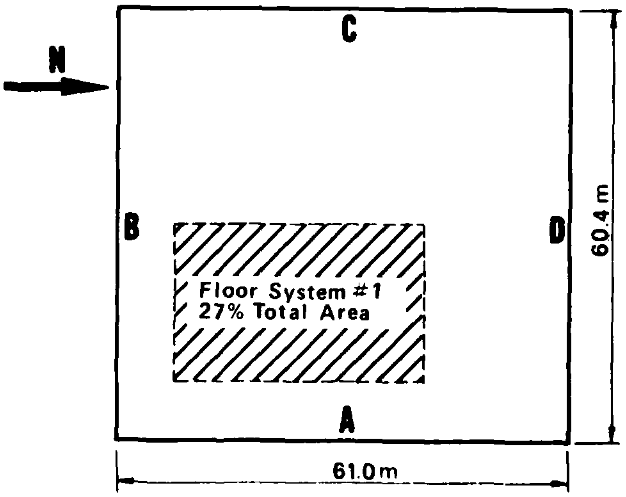


Figure 34: BLDG 245, PORTLAND HILTON HOTEL, Portland, OR - Photograph and Plan of First Floor

B.10.2 Floor Systems

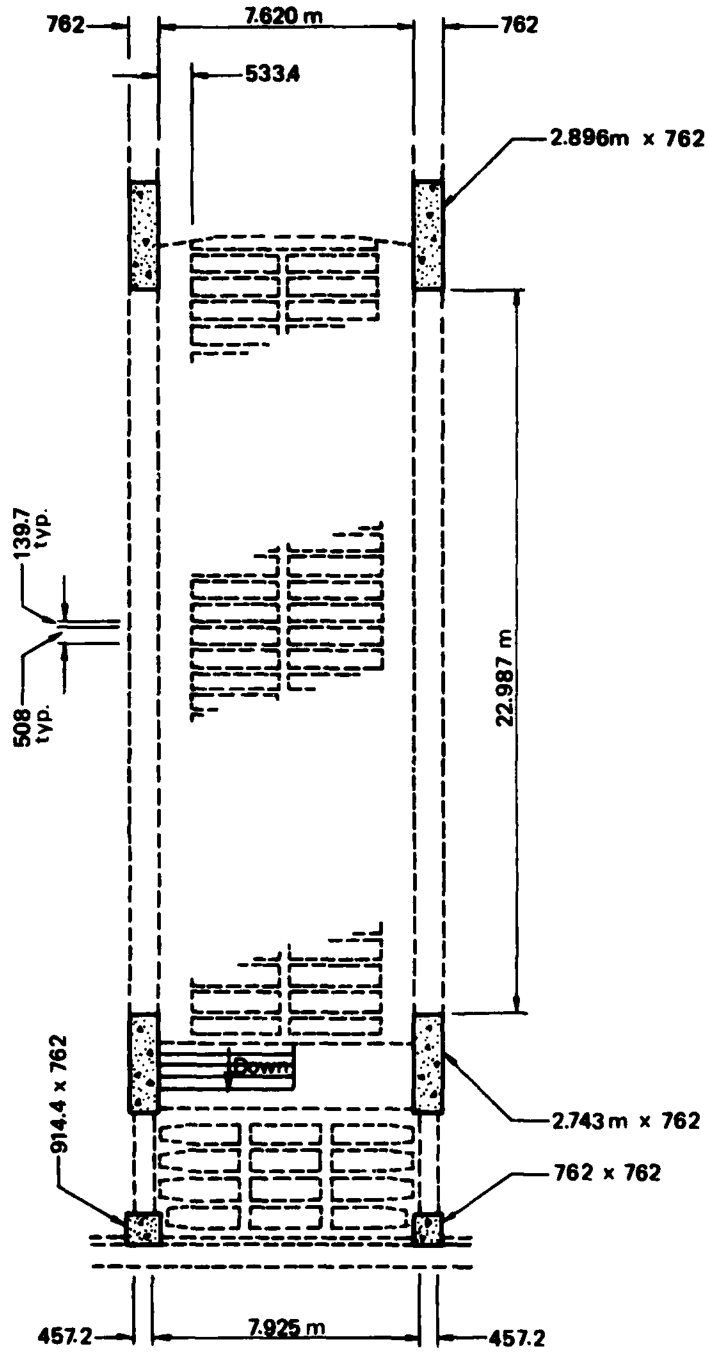
One floor system was chosen for examination, a pan-joist system with a slab, T-beam (joist) and girder. The location of the system and percentage of the total usable basement shelter area it represents are shown in Figure 34. A diagram of the system is shown in Figure 35.

The slab portion of the system was analyzed at 181.5 kPa (26.3 psi) using the unrestrained slab model and 1126.7 kPa (163.4 psi) using the fully restrained model. As noted previously, the restrained model prediction is felt to more closely represent the real strength of the slab. The MICO for the joist was 73.4 kPa (10.7 psi), and for the girder it was 67.4 kPa (9.8 psi). The floor system strength was, therefore, 67.4 kPa (9.8 psi) based on the girder.

B.10.3 Upgrading Potential

The restrained "as built" prediction for the slab is taken as its upgraded value since no upgrade scheme was devised for slabs in a pan-joist system. The joist was upgradable to 273.5 kPa (39.7 psi) by adding support columns at third-spans. The girder reached its maximum potential of 564.5 kPa (81.9 psi) with the scenario of added columns at quarter-span intervals. The upgrading potential of the building basement was determined by the joist at 273.5 kPa (39.7 psi).

Two other scenarios tried were: an added support column at mid-span of the girder along with an added girder supporting the beams of the original girder, 3H; and two added girders supporting the beams, but not the slabs, of the original girder, 3J. These two cases produced predictions of 127.8 and 171.7 kPa (18.5 and 24.9 psi), respectively.



ALL DIMENSIONS IN mm
UNLESS OTHERWISE NOTED

Figure 35: BLDG 245, PORTLAND HILTON HOTEL, Floor Systems

Appendix C

WES UPGRADING TESTS

This appendix is included to show comparisons between the analytical procedures used herein and tests performed by the U.S. Army Engineer Waterways Experiment Station (WES) for FEMA in 1979 and 1980 on floor systems typical of NSS buildings. The intent of the WES tests was to measure response of the "as built" floor system against upgraded versions to see if the load-carrying capacity of conventional commercial buildings could be raised to a level sufficient to withstand peak air-blast overpressures of 137.9 to 344.7 kPa (20 to 50 psi). Two floor systems were tested: slab-and-beam and waffle slab.

The first two sections of the appendix present the WES test program data and comparisons with the single-degree-of-freedom (SDOF) calculated responses. The final section gives the conclusions.

C.1 SLAB-AND-BEAM SYSTEM

C.1.1 Description

One of the most common floor systems found in NSS buildings is a reinforced concrete slab-and-beam construction. Figure 36 shows a diagram typical of this type of floor system. WES decided to test the upgrading potential of this system type.

In order to test under boundary conditions as close to real life as possible, a section of the floor system was tested rather than individual elements. Additionally, the reaction structure was designed so that the boundary conditions reflected, as closely as possible, those of a continuous floor system. A diagram of the final floor design and structural details are shown in Figures 37 through 40.

The system chosen had two slabs with a clear span of 1.88 m in the short direction and 4.01 m in the long direction. The slab was 101.6 mm thick with a span-to-thickness ratio of 18.5. The effective depth of steel reinforcement was 76.2 mm. Longitudinal compressive and tensile reinforcement was provided by #3 bars spaced at 203.2 mm and 304.8 mm, respectively. Transverse tensile reinforcement came from #3 bars spaced at 457.2 mm.

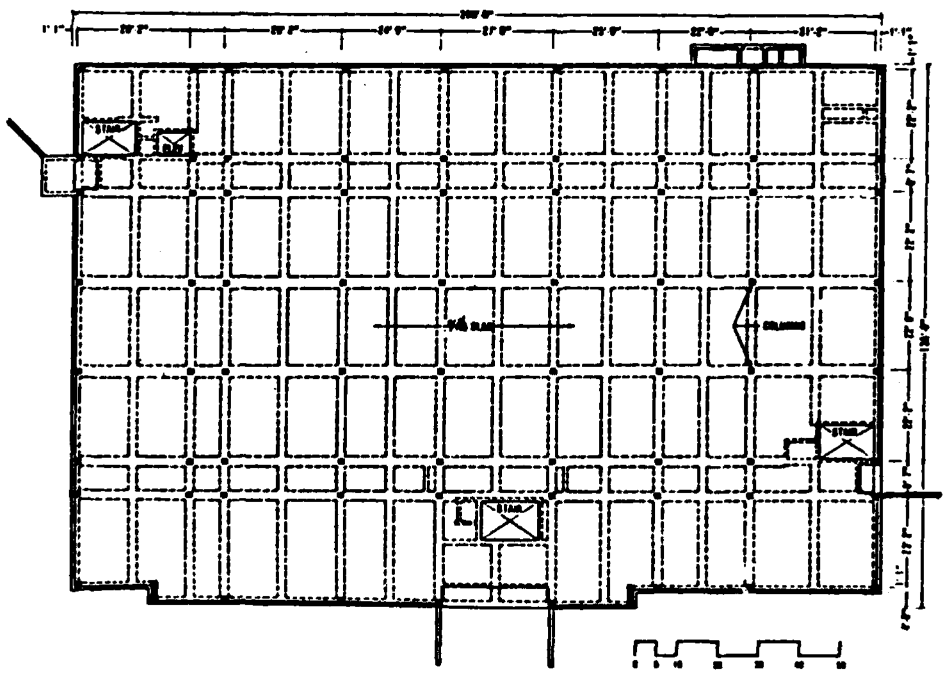


Figure 36: TYPICAL SLAB-AND-BEAM FLOOR SYSTEM

The slabs were supported at the edges by three reinforced concrete beams. Beam details are given in Figures 38, 39, 40.

C.1.2 Upgrading

Several upgrading schemes were considered, but were reduced to two basic methods: (1) using posts, and (2) using beams and posts. These were translated by WES into the following two upgrading designs.

1. The use of 4x4 nominal (88.9 mm x 88.9 mm actual) posts to make different kinds of larger columns to use as a support under the mid-span of the slabs and at intervals under the beams. The layout of this method is given in Figure 41.

2. The use of short lengths of steel beams propped up by steel pipes as a support at the mid-span of the slabs. The layout of this method is given in Figure 42.

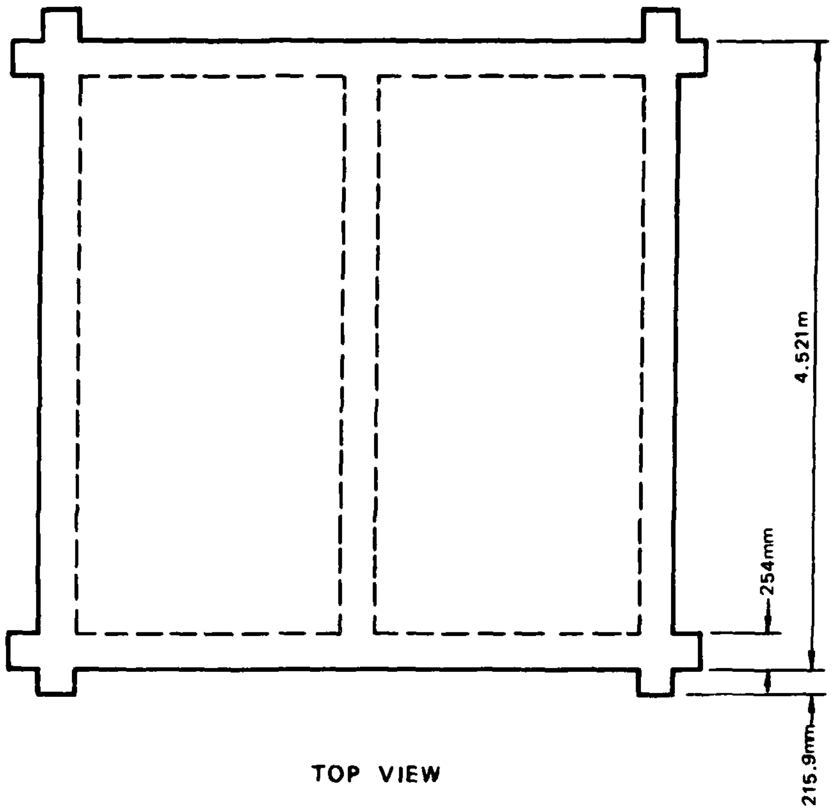
C.1.3 Test Results

The three tests, "as built" (Figures 37 - 40), upgraded using wooden posts (Figure 41), and upgraded using steel beams (Figure 42), were conducted in a large blast load generator. The sides of the structures tested were partially restrained from lateral motion and rotation by the support structure of the blast loading machine.

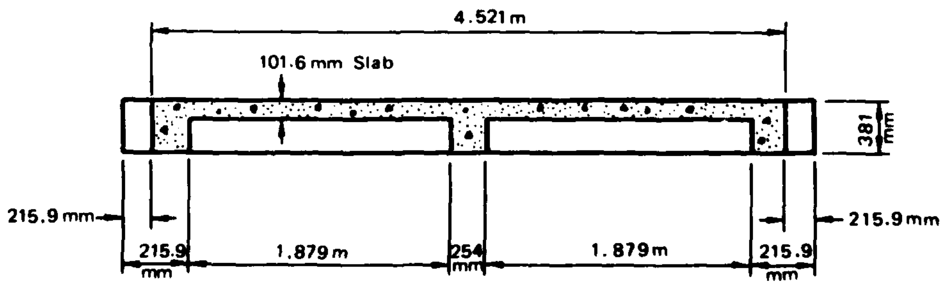
Test 1, the "as built" case, was loaded twice. The first loading, 1A, at 97 kPa (14 psi) peak load, caused large deflections and cracking along all beams. Failure was judged imminent and the second loading, 1B, was designed for 110 kPa (16.0 psi) peak. Use of the wrong primacord, however, resulted in a 228 kPa (33.1 psi) peak loading and approximately 75 kPa (10.9 psi) average load, which collapsed the slab.

Test 2, involving the wood post upgraded system, began with a loading of 228 kPa (33 psi) peak. It was loaded five times (2A, 2B, 2C, 2D, and 2E) at progressively higher overpressures until the slab began to sustain severe cracking above the support posts at a loading of 814 kPa (118 psi) peak pressure [average load of 550 kPa (79.8 psi)].

Test 3, the steel beam upgraded system, was loaded at 228, 448 and 690 kPa (33, 65, and 100 psi). The last loading, 3C, was determined to cause collapse.



TOP VIEW



CENTERLINE SECTION

Figure 37: DIAGRAM OF "AS BUILT" FLOOR SYSTEM - WES TEST 1

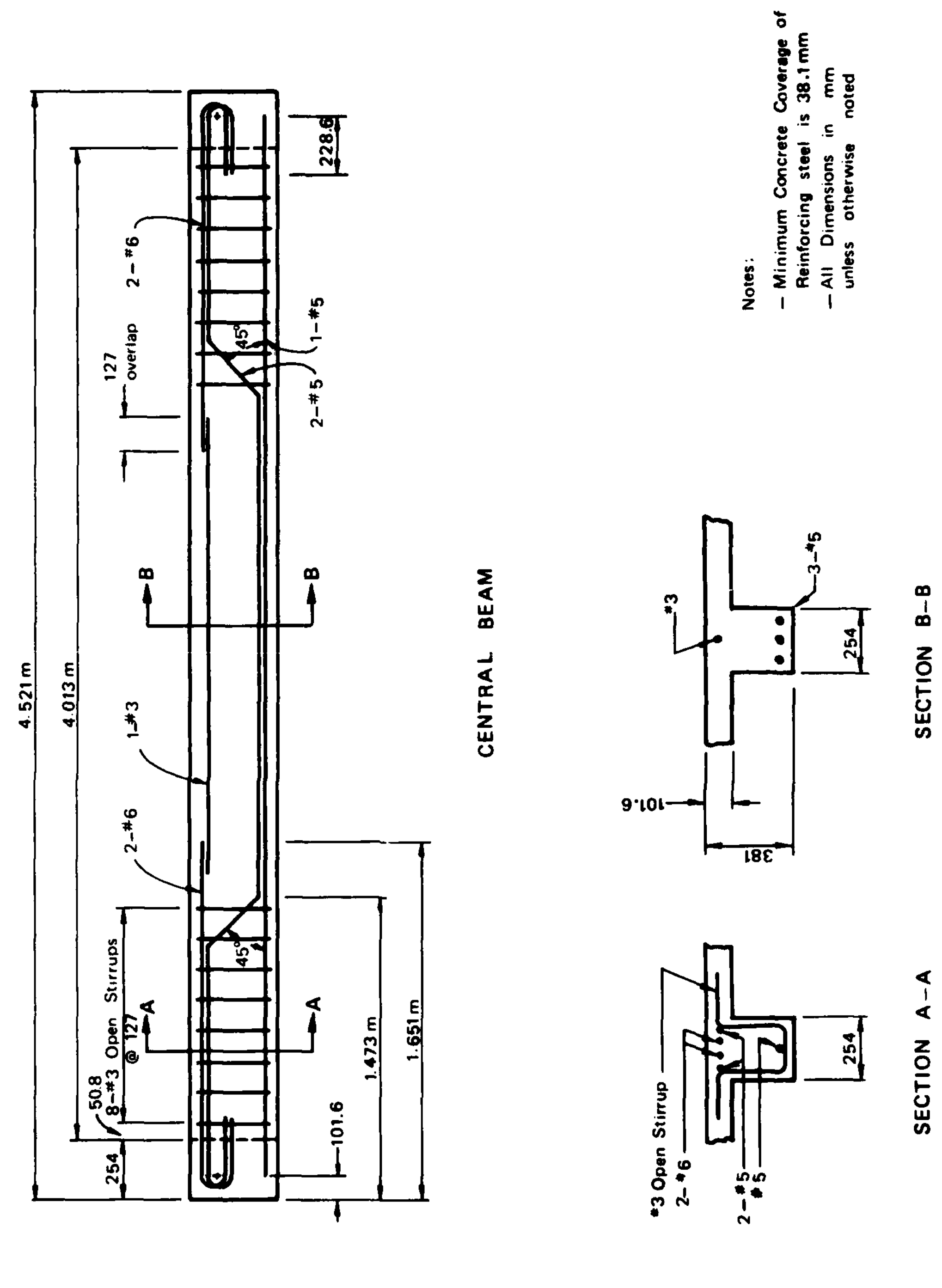


Figure 38: STRUCTURAL DETAILS OF CENTER BEAM - WES TEST 1

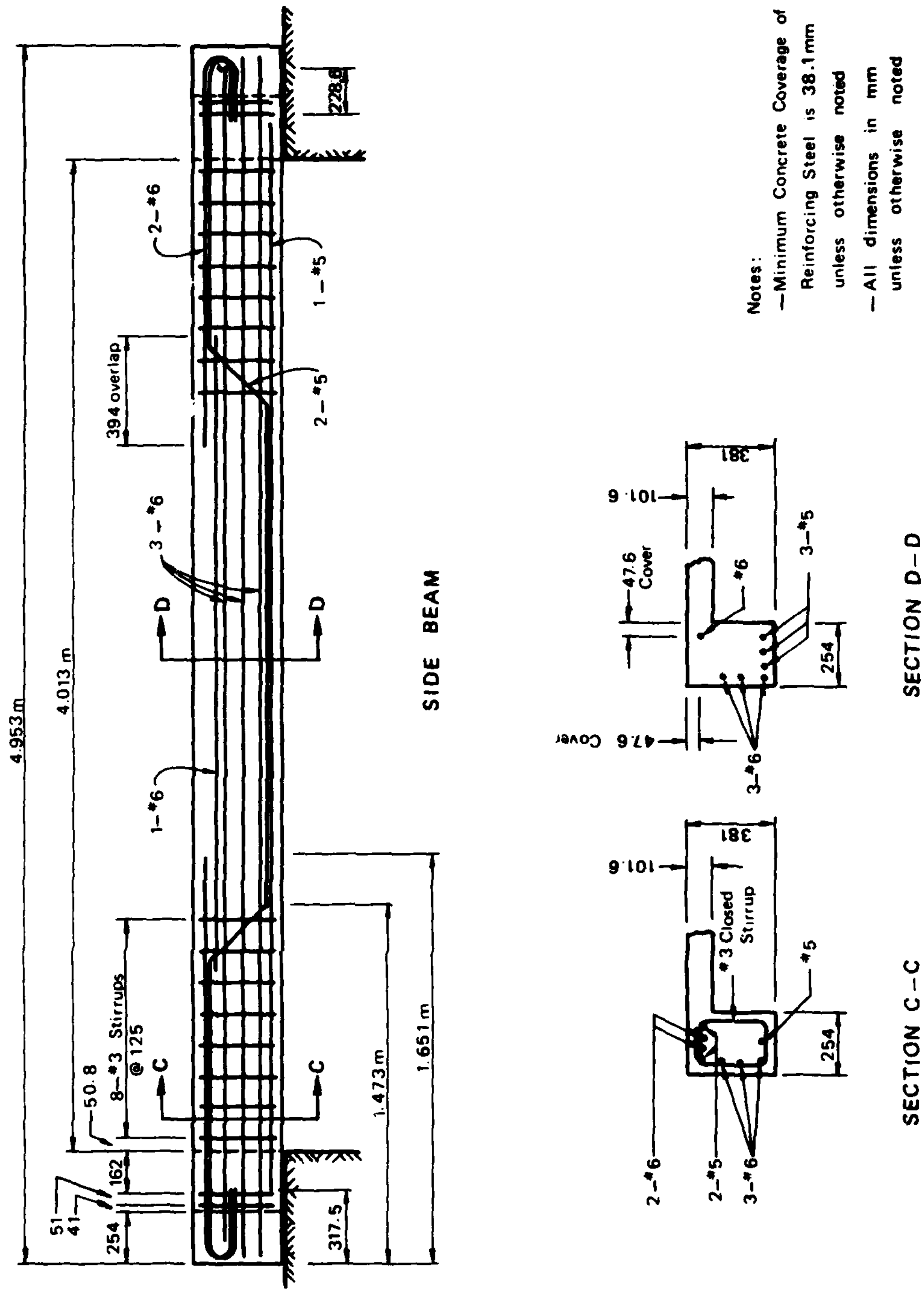


Figure 39: STRUCTURAL DETAILS OF SIDE BEAM - WES TEST 1

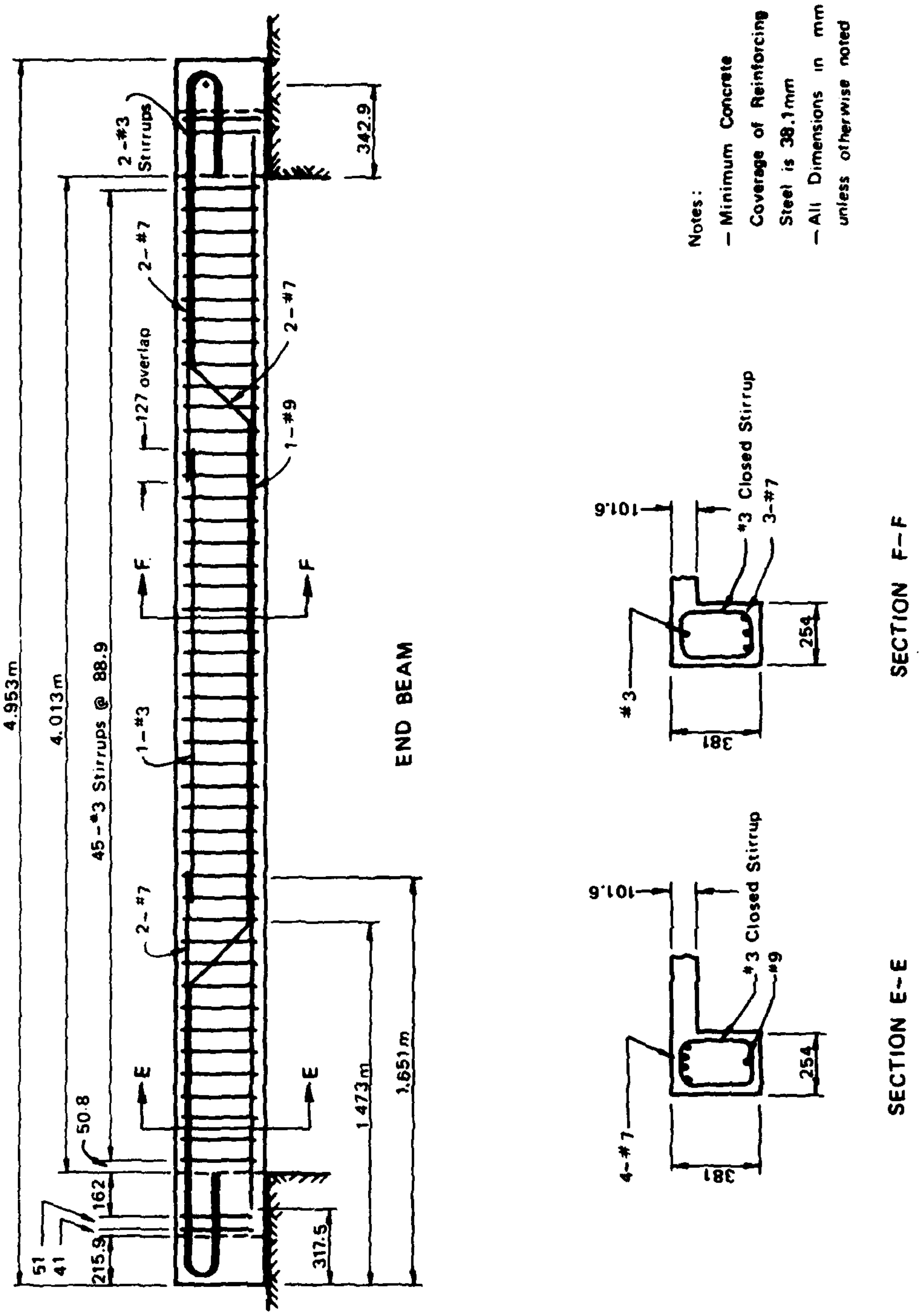
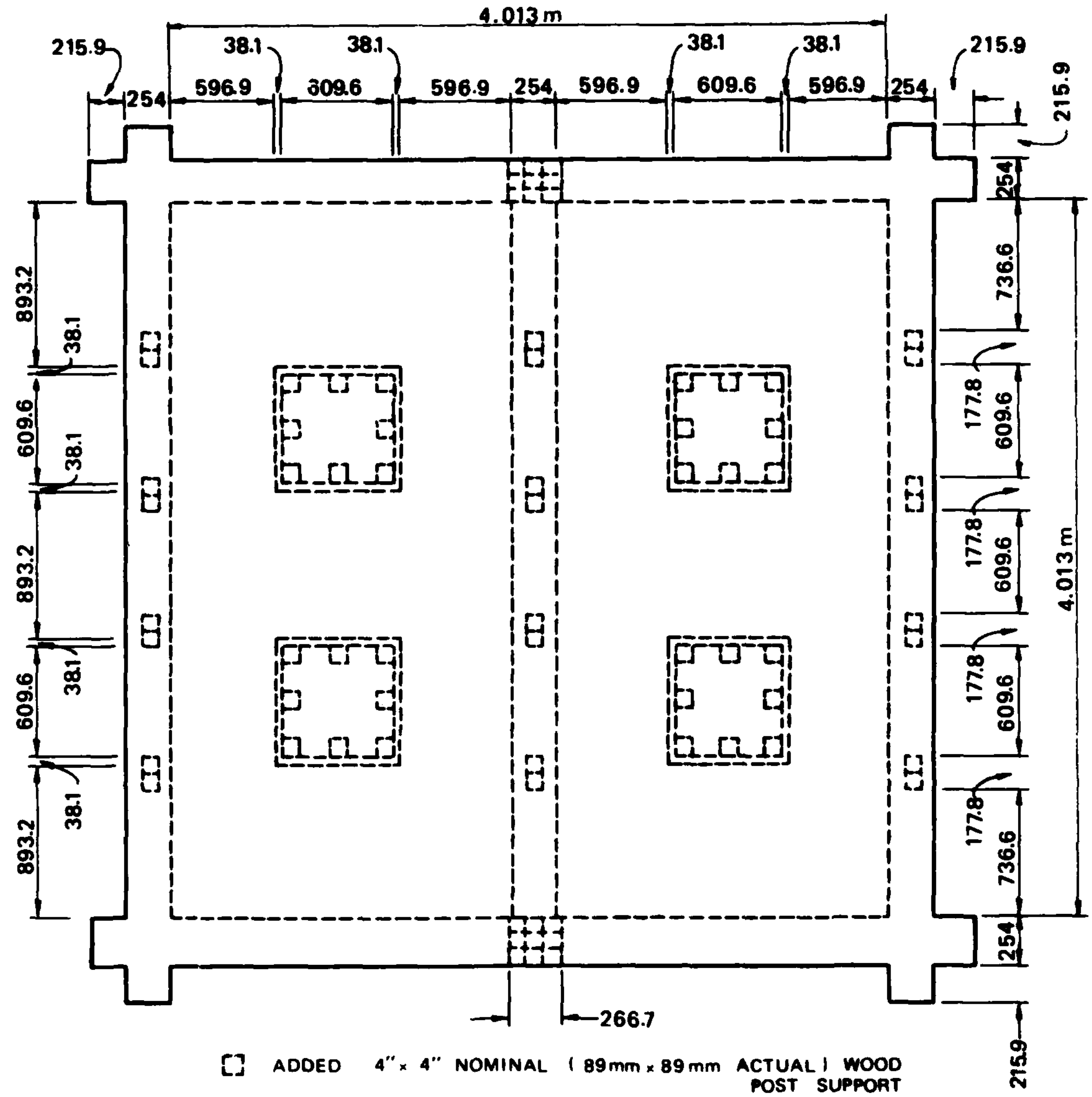
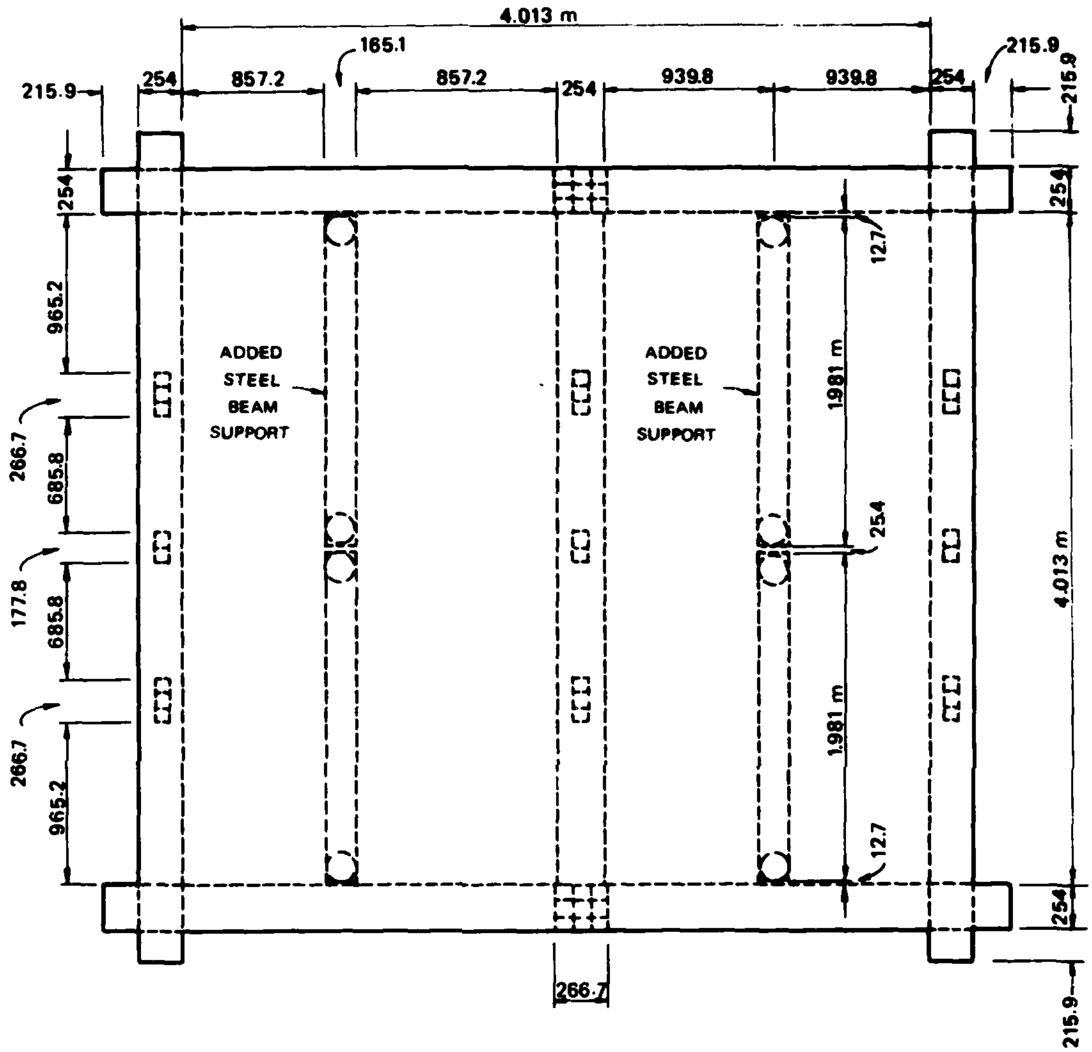


Figure 40: STRUCTURAL DETAILS OF END BEAM - WES TEST 1



All dimensions in millimeters unless otherwise noted

Figure 41: DIAGRAM OF WOOD POST UPGRADE SCHEME - WES TEST 2



ADDED 4" x 4" NOMINAL (89 mm x 89 mm ACTUAL) WOOD POST SUPPORT

ADDED STEEL PIPE SUPPORT

All dimensions in millimeters unless otherwise noted

Figure 42: DIAGRAM OF STEEL BEAM UPGRADE SCHEME - WES TEST 3

C.1.4 Correlation with the SDOF Model

C.1.4.1 "As Built"

Prior to the actual test an analysis was conducted on the "as built" case using the SRI SDOF model. This analysis was based on information received by telephone conversation with WES personnel without benefit of drawings. The pretest predictions (based on failure of slab only) was 110 kPa (16 psi). Actual collapse of the "as built" system was estimated to be very near this value. This prediction appears to have been a fortunate first try.

After the test information was received from WES regarding tested material properties and actual loads applied, the MICO for the system was calculated similarly to the NSS building analysis. Each element of the system was analyzed independently. No interaction was assumed between elements.

The post shot analysis estimated that failure of the beam would control the system strength. The strength thus predicted for the system was 70 kPa (10.2 psi).[3] This value is 36.5 percent lower than the test value. The calculated value is based on the assumption that the central beam and the edge beams would have about the same stiffness (as in a real floor system). The model tested, however, had edge beams with about one-half the loaded area of the central beam, with the result that the edge beams were approximately twice as strong as expected. This extra stiffness of the edge beams would "pick-up" part of the load carried by the central beam if all the beams had the same flexibility. Therefore, it would be expected that the WES beam test strength would be as much as 1.5 times greater than that predicted, or 105 kPa (15.2 psi). Taking this into account the strength of the system can be reevaluated based on the strength of the slab. If no end restraint is assumed, then the MICO of 96 kPa (13.9 psi) is obtained. If the slab were assumed to be fully restrained from horizontal in-plane motion, the MICO would be 217 kPa (31.5 psi). Because of the way in which the test system was constructed, the slab would not be expected to be fully restrained, and since only small edge displacements tend to make the element act as if not restrained, the lower value of 96 kPa is more likely. Therefore, based on the geometry of the tested model, failure would be estimated between 96 and 105 kPa. This value is very close to the test results.

In an actual floor system, because of the larger loading on the beams, collapse would be estimated at approximately 70 kPa (10.2 psi).

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[3]The incipient collapse overpressures given are mean (50 percent probability of failure) values. To estimate 10 and 90 percent probability of failure, multiply this value by .65 and 1.35, respectively.

C.1.4.2 Wood Post Upgrade

The wood post upgraded slab was similarly analyzed by individual elements. The central beam was found to have a MICO 1603 kPa (232.5 psi) and the slab 1163 kPa (168.7 psi) based on the assumption that the supports would not move laterally or rotate. Because parameters for vertical motion of the support and localized failure due to punching (as reported in the test) are not part of the present computer program, the MICO of the upgraded slab (and simulated floor system) would have been predicted at less than 1163 kPa (168.7 psi). Using this number as a base, one finds that the predicted failure pressure is close (within 14 percent) to the predicted collapse overpressure and that it does fall between the estimated 10 and 90 percent probability of failure limits, 758 and 1525 kPa (109.9 and 221.2 psi), discussed in the previous footnote 3.

C.1.4.3 Steel Beam Upgrade

Finally the analysis of the steel beam upgraded slab was made. Again the system was analyzed by individual elements. The beam was calculated to have a MICO of 665 kPa (96.5 psi) and the slab was predicted to have a MICO of 1256 kPa or 376 kPa (182.2 psi or 54.5 psi) depending upon whether the slab was assumed to be fully restrained or not restrained against in-plane motion. Since full restraint is unlikely for the entire upgraded slab, the strength of the slab could be estimated at between 1256 and 376 kPa. For the upgraded system a collapse prediction would probably be based on the strength of the beam since the slab data are inconclusive. Therefore, a MICO of 665 kPa (96.5 psi) would be predicted with 10 and 90 percent values estimated to be 432 and 897 kPa (67.7 and 130.1 psi), respectively. For an actual floor system, where full restraint would be expected on interior panels, the prediction is 665 kPa. On exterior panels, one would expect a MICO of 376 kPa (54.5 psi).

The analyses performed and the results are given in Table 7. The estimates of incipient collapse test overpressures given here support the usefulness of the computer programs which have been developed for FEMA. The test examples show, however, that a great deal of judgment is necessary in predicting MICOs when supports are of a non-standard nature as in the WES tests. Also it is important to note in the cases shown that if the MICOs had been calculated without regard to the peculiarities of the test system, the predictions would have consistently underestimated test values.

TABLE 7

SUMMARY OF WES TEST ANALYSES

Analysis Cases[1]	Incipient Collapse Overpressure		
	Experimental[2] (kPa)	Analytical	
		Unrestrained (kPa)	Restrained (kPa)
As Built	110 (97-228)		
Slab (Preshot)		110	---
Slab		96	216
Beam		70	---
Wood Post Upgrade	1000 (>814)		
Slab		174	1167
Beam		1603	---
Steel Beam Upgrade	600 (450-690)		
Slab		265	1256
Beam		665	---

[1]Analysis was performed after the test shot except where noted.
[2]The incipient collapse overpressure was estimated based on test results. The actual test values bounding the estimated incipient collapse overpressure are given in parentheses. The values in this column are for the tested system.

C.2 WAFFLE SLAB SYSTEMS

In another phase of the WES test program for FEMA, WES examined 19-in. and 30-in. waffle slab systems.

C.2.1 19-Inch Waffle

C.2.1.1 Description

In this test the section was made with a standard 19-in. (482 mm) pan 8 in. (203 mm) deep. The slab portion had a clear span of 432 mm (17 in.) in both directions and was 76 mm (3 in.) thick with 6x6 - W2.9xW2.9 welded wire fabric (WWF) as reinforcement at slab middepth. The element was formed monolithically with the lateral supports and therefore was fully restrained against horizontal motion. The reported concrete and steel strengths were f'_c = 36.3 MPa (5270 psi) and f_y = 483 MPa (70,000) psi).

C.2.1.2 Test Results and SDOF Correlation

Experimentally the slab was observed to have a peak static resistance of 6.2 MPa (900 psi). The SDOF calculated resistance was 5.8 MPa (842 psi). Further calculations predicted that the element could resist a peak load of 4.9 MPa (710 psi) under dynamic loading from a 1-Mt nuclear blast wave form. The test and the analytical data show that the slab portion of a waffle floor system can be expected to have a resistance well in excess of 350 kPa (≈50 psi), one of the values desired in upgrading.

C.2.2 30-Inch Waffle

The maximum resistance and MICO for a 1-Mt nuclear loading were also calculated for the slab portion of a floor system made with 30-in. (762 mm) waffles. The slab was assumed to be 76 mm (3 in.) thick with a free span of 686 mm (27 in.) in both directions. The concrete strength f'_c was taken as 33.1 MPa (4800 psi) and the steel strength f_y was taken as 448 MPa (65,000 psi). The slab was assumed to be reinforced at slab middepth with 6x6 - W2.9xW2.9 WWF. Using these parameters the maximum resistance calculated was 1740 kPa (253 psi) and the MICO was 1440 kPa (209 psi).

C.3 CONCLUSIONS

The analytical results were found to be consistent with the test data. The results did, however, point out the necessity to further study the interaction between elements so that less judgment is needed when predicting the strength of an actual system. This might require development of multiple-degree-of-freedom models or inclusion of yielding supports in the present model.

Appendix D

EXTERIOR BASEMENT WALL ANALYSIS

As part of another FEMA project, under H.L. Murphy Associates, analyses were performed on an exterior basement wall case to determine the effect of the soil mass on the wall response.

The configuration given in Figure 43 was chosen for analysis. The soil triangle, defined by a 45-degree angle starting at the bottom of the basement floor to the top of the basement slab (see Figure 43), was assumed to respond with the wall (i.e., the effective mass of the wall was the sum of the soil mass and the wall mass).

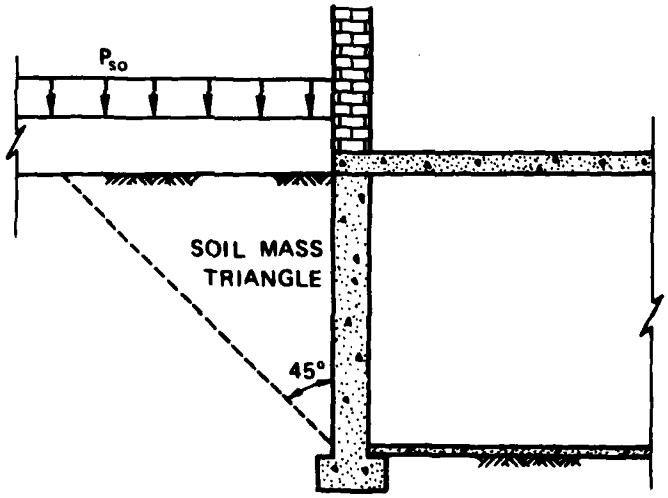

Figure 43: EXTERIOR BASEMENT WALL

Various coefficients of horizontal to vertical load were chosen for analysis (100%, 50% and 15%). However, preliminary analysis showed that the predicted MICO was inversely proportional to the reciprocal of the horizontal/vertical load coefficient; only the 100 percent coefficient

was used in all calculations thereafter. To determine the MICOs at other ratios, simply multiply by the reciprocal of the horizontal/vertical load coefficient [i.e., if horizontal/vertical load equals 50 percent (.5), multiply the value calculated at 100 percent by the reciprocal, 2.0]. The soil was conservatively assumed to have a mass of 2.08 Mg/m^3 (130 lb/ft^3). Friction effects along the boundary of the soil mass were ignored. The R/C wall was assumed to have the following properties:

- One-way simply supported response mode action
- Vertical clearspan of 3 m (118 in.)
- Thickness of 305 mm (12 in.)
- Mass of 2.4 Mg/m^3 (150 lb/ft^3)
- Concrete strength, f_c' of 20.7 MPa (3000 psi)
- Dynamic steel strength, f_{dy} of 365.4 MPa (53,000 psi)
- 0.17 percent steel in both faces

The loading was assumed to be a 1-Mt (4.2 PJ) nuclear event on a side wall with a drag coefficient of zero and the time of peak load (rise-time) at zero and 36 ms.

Based upon these assumptions, and using the computer programs previously developed at SRI, the following MICO for various mass and loading rise-time assumptions were computed:

	Zero Rise-Time	36 ms Rise-Time
Wall mass only	48.3 kPa (7.0 psi)	49.6 kPa (7.2 psi)
Wall and soil mass	53.8 kPa (7.8 psi)	55.2 kPa (8.0 psi)

As can be seen from this table, the mass has little effect on the calculated MICO; the wall is insensitive to rise-time variations. Therefore, soil wave speeds have minimal effect on the model. Both effects are principally due to the relatively long duration of the 1-Mt (4.2 PJ) loading and the relatively long natural periods of vibration in relation to the rise-time of the elements being considered.

These calculations were made to determine the sensitivity of the wall model to soil mass interacting with the wall. As can be seen, if the soil mass is assumed to respond with the wall then there is little effect on the predicted MICO. While this sensitivity test is inconclu-

sive in determining the effect of the soil response on the wall deflection, it does show that the motion of the soil mass together with the wall mass would not affect predictions significantly.

Further study should be done on predicting the response of the exterior basement walls, certainly if buildings are to be upgraded to resist blast loads of approximately 350 kPa (50 psi). One possible mechanism of response not examined here is that the soil mass would only move with the wall for a certain horizontal distance before losing contact with the wall (i.e., ending the load on the wall). This would assume the soil mass would have a limited range of horizontal displacement. To determine if this is a possible mechanism of response, it would be necessary to review the literature to see if any buried walls have been tested to incipient collapse, make some further analytical calculations, and probably perform dynamic field tests. In any event, if basements are to be realistically upgraded to resist the loads contemplated, much more must be known about the response of the basement walls.

Appendix E

FLOOR ELEMENT ANALYSES - ENGLISH UNITS

The following table (Table 8) is the same as Table 1 given in the section "Building Analysis of "As Built" Case" except English units are substituted for metric. For a discussion on how to use the table please refer to that section.

TABLE 8

SUMMARY OF FLOOR ELEMENT ANALYSES - ENGLISH UNITS

Case	Element Type	L_s (in.)	L_L (in.)	b_b (in.)	h (in.)	f_c' (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections: 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio: Short	Long	Incipient Collapse Overpressure: Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)
Building 100, U.S. Post Office (RCF)																						
1a	RCS-1	337	1,320		12	3,000	53,000	1	0.0172	0.0013	0.0013	0.0013	--	--	--	--	0.0013	--	5.69	0.63	4.88	6.49
1b	RCS-1	168.5	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	23.06	62.76	619.52	26.60
1c	RCS-1	112.3	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	51.23	4.56	45.38	57.08
1d	RCS-1	84.25	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	91.43	8.63	80.37	102.49
2a	RCS-2	79.25	131		4.5	3,000	53,000	4	0.0051	0.0051	0.0039	0.0	0.0048	0.0048	--	--	0.0051	--	*13.64	1.62	11.57	15.72
2e	RCS-2	79.25	65.5		4.5	3,000	53,000	4	0.0051	0.0051	0.0039	0.0	0.0048	0.0048	--	--	0.0051		16.92	1.97	14.40	19.44
3A	RCB-2	197		40.13	16.5	3,000	53,000	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	*18.35	2.46	15.19	21.51
3B	RCB-2	98.5		40.13	16.5	3,000	53,000	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	*39.01	5.38	32.12	45.90
4a	RCS-3	289	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	5.83	0.62	5.03	6.62
4b	RCS-3	144.5	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	25.60	2.67	22.18	29.03
4c	RCS-3	96.3	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	59.47	6.82	50.72	68.22
4d	RCS-3	72.25	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	107.89	12.07	92.43	123.36
5a	RCS-4	17.67	82.5		3	3,000	53,000	6	0.0072	0.0	--	--	0.0072	0.0	--	--	0.0072	--	41.55	4.49	35.80	47.30
5a/r	RCS-4	17.67	82.5		3	3,000	53,000	n/a	0.0072	0.0	0.0012	0.0	0.0072	0.0	0.0012	0.0	0.0072	--	188.13	18.95	168.83	212.42
6A	RCJ-4	360		7.28	17	3,000	53,000	5T	0.0203	0.0	--	--	--	--	--	--	--	--	3.51	0.51	2.86	4.17
6B	RCJ-4	180		7.28	17	3,000	53,000	5T	0.0203	0.0	--	--	--	--	--	--	--	--	15.82	1.98	13.28	18.35
6C	RCJ-4	120		7.28	17	3,000	53,000	5T	0.0203	0.0	--	--	--	--	--	--	--	--	34.47	3.30	30.25	38.70
6D	RCJ-4	90		7.28	17	3,000	53,000	5T	0.0203	0.0	--	--	--	--	--	--	--	--	63.35	7.43	53.82	72.87
7a	RCS-5	223	2,328		10	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	5.16	0.72	4.24	6.08
7b	RCS-5	111.5	2,328		10	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	22.72	2.83	19.09	26.35
7c	RCS-5	74.3	2,328		10	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	50.33	4.68	44.33	56.33
Building 110, Henry R. Landis State Hospital (RCF)																						
1a	RCS-1	171	1,552		7	3,300	53,000	7	0.0074	0.0	--	--	0.0087	0.0	--	--	--	--	5.06	0.54	4.37	5.75
1b	RCS-1	85.5	1,552		7	3,300	53,000	5	0.0074	0.0	--	--	--	--	--	--	--	--	13.56	1.34	11.84	15.27
2A	RCB-1	264		18	20	3,300	53,000	6	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	4.15	0.57	3.42	4.88
2B	RCB-1	132		18	20	3,300	53,000	7	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	12.58	1.22	11.02	14.14
2C	RCB-1	88		18	20	3,300	53,000	5	0.0127	0.0	--	--	--	--	--	--	--	--	17.46	2.04	14.85	20.07
3a	RCS-2	174.5	254		6	3,300	53,000	3	0.0037	0.0	0.0030	0.0	--	--	0.0042	0.0027	--	--	3.23	0.61	2.46	4.01
3b	RCS-2	87.25	254		6	3,300	53,000	1	0.0037	0.0	0.0030	0.0	--	--	--	--	--	--	6.77	0.94	5.56	7.97
4A	RCB-2	166		22	14	3,300	53,000	7	0.0093	0.0	--	--	0.0157	0.0094	--	--	--	--	5.97	0.67	5.12	6.83
4B	RCB-2	83		22	14	3,300	53,000	5	0.0093	0.0	--	--	--	--	--	--	--	--	14.09	1.37	12.32	15.85
5a	RCS-3	114	256		4	3,300	53,000	6	0.0111	0.0	--	--	0.0097	0.0056	--	--	0.0056	--	* 7.79	0.83	6.73	8.86
6A	RCB-3	260		20	12	3,300	53,000	6	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	* 4.63	0.66	3.78	5.49
6B	RCB-3	130		20	12	3,300	53,000	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	13.16	1.31	11.49	14.83
6C	RCB-3	86.67		20	12	3,300	53,000	5	0.0291	0.0	--	--	--	--	--	--	0.0099	--	17.06	1.56	15.06	19.06
6E	RCB-3	130		20	12	3,300	53,000	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0099	--	15.85	1.26	14.23	17.46
7A	RCG-3	195		28	14	3,300	53,000	6	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	4.20	0.48	3.59	4.81
7B	RCG-3	97.5		28	14	3,300	53,000	7	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	13.97	1.44	12.12	15.81

TABLE 8

SUMMARY OF FLOOR ELEMENT ANALYSES - ENGLISH UNITS (continued)

Case	Element Type	L_s (in.)	L_L (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio Short	Long	Incipient Collapse Overpressure Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)
Building 111, Grant Building (STF)																						
1a	RCS-1	18.33	--		2.5	2,000	44,000	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	*14.61	0.69	13.73	15.50
1a/r	RCS-1	18.33	--		2.5	2,000	44,000	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	67.59	6.87	58.79	76.40
2A	RCJ-1	216		5.21	12.5	2,000	44,000	7T	0.0237	0.0	--	--	0.0270	0.0205	--	--	0.0135	--	7.27	0.70	6.37	8.16
2B	RCJ-1	108		5.21	12.5	2,000	44,000	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	15.13	1.44	13.28	16.98
2C	RCJ-1	72		5.21	12.5	2,000	44,000	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	34.28	3.43	29.89	38.67
3A	STG-1	218		12.04	16.1		41,000	6	(S=144.88 in³, 16CI83)										10.97	1.60	8.92	13.02
3B	STG-1	109		12.04	16.1		41,000	6	(S=144.88 in³, 16CI83)										39.25	4.37	33.65	44.85
3C	STG-1	72.67		12.04	16.1		41,000	6	(S=144.88 in³, 16CI83)										90.83	12.86	74.35	107.32
Building 136, First Federal Savings & Loan Assn. (RCF)																						
1a	RCS-1	18	82		2	3,000	56,000	6	0.0036	0.0	--	--	0.0036	0.0	--	--	0.0036	--	*12.40	1.20	10.86	613.93
1a/r	RCS-1	18	82		2	3,000	56,000	n/a	0.0036	0.0	0.0040	0.0	0.0036	0.0	0.0040	0.0	0.0036	--	70.86	6.12	63.01	78.70
2A	RCJ-1	168		6.18	14	3,000	53,000	7T	0.0039	0.0039	--	--	0.0043	0.0040	--	--	--	--	3.53	0.49	2.90	4.16
2B	RCJ-1	84		6.18	14	3,000	53,000	5T	0.0039	0.0039	--	--	--	--	--	--	--	--	9.70	1.23	8.13	11.28
3A	RCG-1	228		42	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	4.29	0.56	3.58	5.00
3B	RCG-1	114		42	14	3,000	53,000	7	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	14.00	1.65	11.88	16.11
3C	RCG-1	76		42	14	3,000	53,000	5	0.0113	0.0113	--	--	--	--	--	--	--	--	20.70	2.07	18.05	23.36
3H	RCG-1	228		42	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	8.50	0.83	7.44	9.57
Building 167, Lafayette Towers Building #2 (RCF)																						
1a	RCFP-1	240	240		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	6.31	0.76	5.33	7.29
1a/s	RCFP-1	240	240		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	1.63	0.01	1.62	1.64
1h	RCFP-1	120	120		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	30.83	3.96	625.75	35.91
1h/s	RCFP-1	120	120		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	8.77	0.02	8.75	8.79
1i	RCS-1	240	240		10	3,000	53,000	2	0.0030	0.0	0.0030	0.0	0.0030	0.0	0.0030	0.0015	--	--	9.13	0.98	7.87	10.39
1j	RCS-1	120	120		10	3,000	53,000	1	0.0030	0.0	0.0030	0.0	--	--	--	--	--	--	19.90	2.59	16.57	23.22
Building 188, State Wildlife Conservation Building (RCF)																						
1a	RCS-1	192	404		6	3,000	44,000	4	0.0094	0.0	0.0038	0.0	0.0142	0.0025	--	--	0.0094	--	*10.52	1.40	8.72	12.31
1b	RCS-1	81	404		6	3,000	44,000	1	0.0094	0.0	0.0038	0.0	--	--	--	--	0.0094	--	*20.86	2.71	17.38	24.34
2A	RCB-1	158		48	15.5	3,000	44,000	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	7.46	0.76	6.48	8.43
2B	RCB-1	79		48	15.5	3,000	44,000	5	0.0086	0.0	--	--	--	--	--	--	0.0051	--	23.42	2.29	20.49	26.36
2H	RCB-1	158		48	15.5	3,000	44,000	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	11.34	1.14	9.88	12.80
2I	RCB-1	79		48	15.5	3,000	44,000	5	0.0086	0.0	--	--	--	--	--	--	0.0051	--	14.92	1.45	13.05	16.78
3A	RCB-1	224		48	15.5	3,000	44,000	7	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0080	--	* 4.47	0.56	3.75	5.18
3B	RCB-1	112		48	15.5	3,000	44,000	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	15.51	1.47	13.62	17.40
3C	RCB-1	74.67		48	15.5	3,000	44,000	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	24.49	2.93	20.74	28.24

TABLE 8

SUMMARY OF FLOOR ELEMENT ANALYSES - ENGLISH UNITS (continued)

Case	Element Type	L_s (in.)	L_L (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections								Tensile Membrane Steel Ratio		Incipient Collapse Overpressure			
									1		2		3		4				Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)
									p	p'	p	p'	p	p'	p	p'	Short	Long				
Building 200, Fitzsimmons General Hospital (RCF)																						
1a	RCS-1	78	266		4	3,000	44,000	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	6.54	0.54	5.85	7.22
1e	RCS-1	78	133		4	3,000	44,000	4	0.0073	0.0	0.0040	0.0	0.0073	0.0073	--	--	--	--	8.77	0.83	7.71	9.84
1h	RCS-1	78	88.7		4	3,000	44,000	4	0.0073	0.0	0.0040	0.0	0.0073	0.0073	--	--	--	--	10.38	1.17	8.88	11.87
2A	RCB-1	266		12	18	3,000	44,000	6	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	* 4.79	0.56	4.07	5.52
2B	RCB-1	133		12	18	3,000	44,000	7	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	13.86	1.67	11.72	15.99
2C	RCB-1	88.67		12	18	3,000	44,000	5	0.0138	0.0	--	--	--	--	--	--	0.0138	--	17.39	2.01	14.81	19.98
3A	RCG-1	255		16	28	3,000	44,000	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	4.92	0.52	4.25	5.60
3B	RCG-1	127.5		16	28	3,000	44,000	7	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	17.09	2.15	14.32	19.85
3C	RCG-1	85		16	28	3,000	44,000	5	0.0101	0.0	--	--	--	--	--	--	0.0025	--	28.50	3.46	624.07	32.93
3N	RCG-1	255		16	28	3,000	44,000	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	8.55	0.96	7.31	9.78
4a	RCS-2	108	180		4	3,000	44,000	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	3.28	0.30	2.90	3.66
4e	RCS-2	90	108		4	3,000	44,000	3	0.0040	0.0	0.0073	0.0	--	--	0.0073	0.0073	--	--	6.06	0.73	5.13	6.99
5A	RCB-2	177		12	16	3,000	44,000	6	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	* 3.94	0.62	3.15	4.73
5B	RCB-2	88.5		12	16	3,000	44,000	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	11.99	1.16	610.50	13.47
5E	RCB-2	88.5		12	16	3,000	44,000	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	18.65	2.23	15.80	21.51
6A	RCB-2	105		12	20	3,000	44,000	6	0.0041	0.0029	--	--	0.0029	0.0041	--	--	0.0070	--	12.17	0.74	11.22	13.12
Building 220, Fidelity Federal Plaza Building (RCF)																						
1a	RCS-1	95	306		4.5	3,000	53,000	6	0.0043	0.0	--	--	0.0043	0.0021	--	--	0.0043	--	* 8.30	1.08	6.92	9.68
1e	RCS-1	95	153		4.5	3,000	53,000	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	* 8.99	0.99	7.72	10.26
1f	RCS-1	95	102		4.5	3,000	53,000	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	* 9.80	0.98	8.55	11.06
2A	RCB-1	306		13	20	3,000	53,000	6	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	* 5.50	0.84	4.42	6.58
2B	RCB-1	153		13	20	3,000	53,000	7	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	15.17	1.82	12.84	17.50
2C	RCB-1	102		13	20	3,000	53,000	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	20.21	1.81	17.89	22.52
2D	RCB-1	76.5		13	20	3,000	53,000	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	36.44	3.25	32.28	40.60
3A	RCG-1	312		18	36	3,000	53,000	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	3.72	0.36	3.26	4.18
3B	RCG-1	156		18	36	3,000	53,000	7	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	12.21	1.25	10.60	13.81
3C	RCG-1	104		18	36	3,000	53,000	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	20.82	2.12	18.11	23.54
3D	RCG-1	78		18	36	3,000	53,000	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	34.84	4.42	29.17	40.51
3H	RCG-1	312		18	36	3,000	53,000	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	7.90	0.94	6.69	9.11
Building 225, Broadway Crenshaw Building (RCF)																						
1a	RCFS-1	288	288		9	2,500	53,000	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	* 7.17	0.99	5.90	8.44
1a/s	RCFS-1	288	288		9	2,500	53,000	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	* 6.88	0.75	5.92	7.85
1i	RCS-1	288	288		9	2,500	53,000	2	0.0044	0.0	0.0044	0.0	0.0044	0.0	0.0054	0.0	0.0027	0.0027	* 8.79	0.99	7.70	10.23
1j	RCS-1	144	144		9	2,500	53,000	1	0.0044	0.0	0.0044	0.0	--	--	--	--	0.0027	0.0027	*18.22	2.56	614.94	21.49
2a	RCS-2	75	286		4	2,500	53,000	5	0.0111	0.0	--	--	--	--	--	--	0.0056	--	*11.29	1.25	9.69	12.89
2e	RCS-2	75	143		4	2,500	53,000	1	0.0111	0.0	0.0046	0.0	--	--	--	--	0.0056	--	*12.61	1.55	10.63	14.60
2f	RCS-2	75	95.3		4	2,500	53,000	1	0.0111	0.0	0.0046	0.0	--	--	--	--	0.0056	--	*12.93	1.38	11.17	14.70
3A	RCB-2	286		16	25	2,500	53,000	7	0.0123	0.0	--	--	0.0126	0.0062	--	--	0.0062	--	7.46	0.94	6.25	8.66
3B	RCB-2	143		16	25	2,500	53,000	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	19.88	1.96	617.38	22.39
3C	RCB-2	95.3		16	25	2,500	53,000	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	46.94	5.27	40.19	53.70
4A	RCG-2	253		26	30	2,500	53,000	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	12.46	61.43	10.62	14.29
4B	RCG-2	126.5		26	30	2,500	53,000	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	30.09	3.18	26.02	34.17
4C	RCG-2	84.3		26	30	2,500	53,000	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	68.62	7.05	59.59	77.66
4H	RCG-2	253		26	30	2,500	53,000	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	25.26	62.90	21.54	28.98

TABLE 8

SUMMARY OF FLOOR ELEMENT ANALYSES - ENGLISH UNITS (concluded)

Case	Element Type	L_s (in.)	L_L (in.)	b_o (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio Short	Long	Incipient Collapse Overpressure Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)
Building 227, May Company Eastland Shopping Center (RCF)																						
1a	RCS-1	29			3	3,000	56,000	6	0.0032	0.0	--	--	0.0032	0.0	--	--	0.0032	--	#10.19	0.66	9.35	11.03
1a/r	RCS-1	29			3	3,000	56,000	n/a	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	--	51.50	5.25	44.77	58.24
2A	RCJ-1	270		7.5	17	3,000	53,000	7T	0.0107	0.0	--	--	0.0107	0.0053	--	--	0.0053	--	3.49	0.40	2.98	4.00
2B	RCJ-1	135		7.5	17	3,000	53,000	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	10.20	1.37	8.44	11.96
2C	RCJ-1	90		7.5	17	3,000	53,000	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	21.91	2.57	18.62	25.21
3A	RCG-1	272		30	24	3,000	53,000	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	7.57	0.72	6.64	8.49
3B	RCG-1	136		30	24	3,000	53,000	7	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	22.87	2.23	20.02	25.73
3C	RCG-1	90.67		30	24	3,000	53,000	5	0.0157	0.0	--	--	--	--	--	--	0.0137	--	30.73	3.29	26.51	34.94
3H	RCG-1	272		30	24	3,000	53,000	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	14.17	1.70	11.99	16.36
4A	RCG-1	280		30	26	3,000	53,000	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	6.73	0.65	5.90	7.56
4B	RCG-1	140		30	26	3,008	53,000	7	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	20.87	2.35	17.86	623.86
4C	RCG-1	93.3		30	26	3,000	53,000	5	0.0118	0.0	--	--	--	--	--	--	--	--	28.90	3.28	24.69	33.10
4H	RCG-1	280		30	26	3,000	53,000	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	14.23	1.63	12.13	16.32
Building 245, Portland Hilton Hotel (RCF)																						
1a	RCS-1	18	126		3	3,000	56,000	6	0.0029	0.0	--	--	0.0029	0.0	--	--	0.0029	--	26.33	0.16	26.12	26.54
1a/r	RCS-1	18	126		3	3,000	56,000	n/a	0.0029	0.0	0.0005	0.0	0.0029	0.0	0.0005	0.0	0.0029	--	163.42	16.31	142.51	184.34
2A	RCJ-1	258		5.5	15	3,000	53,000	6T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	10.65	1.39	8.87	12.43
2B	RCJ-1	129		5.5	15	3,000	53,000	7T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	31.34	3.86	26.39	36.29
2C	RCJ-1	86		5.5	15	3,000	53,000	5T	0.0181	0.0103	--	--	--	--	--	--	--	--	39.66	4.14	34.35	44.97
3A	RCG-1	905		30	76.8	3,000	53,000	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	9.78	1.37	8.03	11.53
3B	RCG-1	452.5		30	76.8	3,000	53,000	7	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	31.53	4.02	26.37	36.68
3C	RCG-1	301.67		30	76.8	3,000	53,000	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	45.89	4.52	40.09	51.69
3D	RCG-1	226.25		30	76.8	3,000	53,000	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	81.88	7.91	71.74	92.02
3H	RCG-1	905		30	76.8	3,000	53,000	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	18.54	2.51	15.32	21.77
3J	RCG-1	905		30	76.8	3,000	53,000	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	24.90	2.67	21.48	28.33

Appendix F

NOTATION

A_s	Area of tension steel in reinforced concrete slab per unit width
A_s	Area of tension steel in reinforced concrete beam
A'_s	Area of compression steel in reinforced concrete slab per unit width
A'_s	Area of compression steel in reinforced concrete beam
b	Width of cross section
b_b	Width of beam cross section
d	Distance from extreme compression fiber to centroid of tension reinforcement
f'_c	Compressive strength of concrete
f_{dy}	Dynamic yield strength of reinforcing steel
h	Thickness of slab or beam
L_ℓ	Length of slab in long direction
L_s	Length of slab in short direction
L_s	Length of beam
p	Ratio of tension reinforcing steel area, A_s/bd
p'	Ratio of compression reinforcing steel area, A'_s/bd

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